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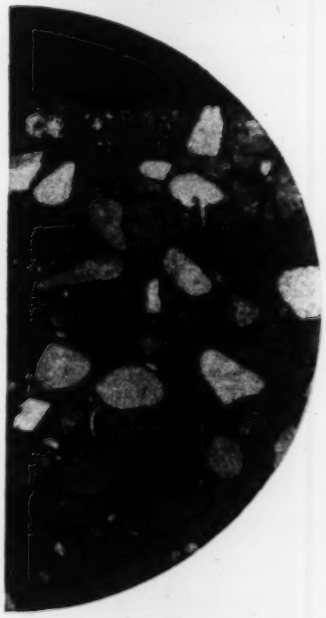

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Engineering News-Record

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Power on the Tennessee River

Federal legislation contemplates a large increase in the developed water power—Army engineers' study of power, navigation and flood control in the entire river basin supplies the basis for the present planning

LEGISLATION authorizing the construction of dams on the Tennessee and Clinch rivers, now before Congress as part of the administration's plan for a coordinated development of the entire Tennessee River basin, raises questions as to what are the power possibilities of that river and its tributaries and what will be the cost of developing the power.

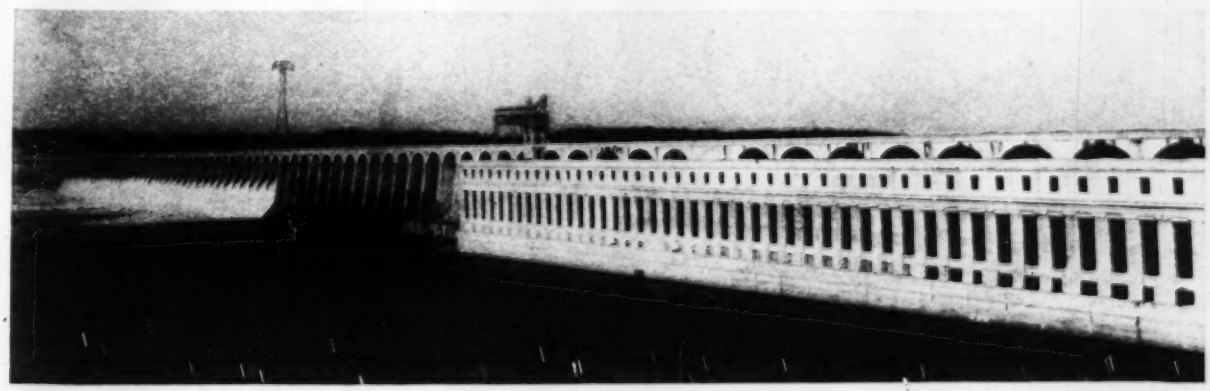
Eight years ago the Corps of Engineers, U. S. Army, under instructions from Congress, undertook to answer these questions, also to restudy the navigation requirements of the river and to determine what could be done to reduce flood damages. The corps' report, prepared in detail under the direction of Major Lewis H. Watkins, district engineer at Chattanooga, was completed early in 1929 and was submitted to Congress three years ago. It has been used as the source for much of the statistical information given herein, as the studies then made are expected to serve as the basis for the power development now contemplated.

The power possibilities of the Tennessee River have long been recognized, but economic considerations have retarded realization of them. Until the federal government undertook the development of power at Muscle Shoals as a war measure, the only plant on the main river was at Hales Bar near Chattanooga. On the tributaries there were a number of small plants at easily developed sites. Since the war a few large plants have been built upon the Little Tennessee and its tributaries by the Aluminum Co.

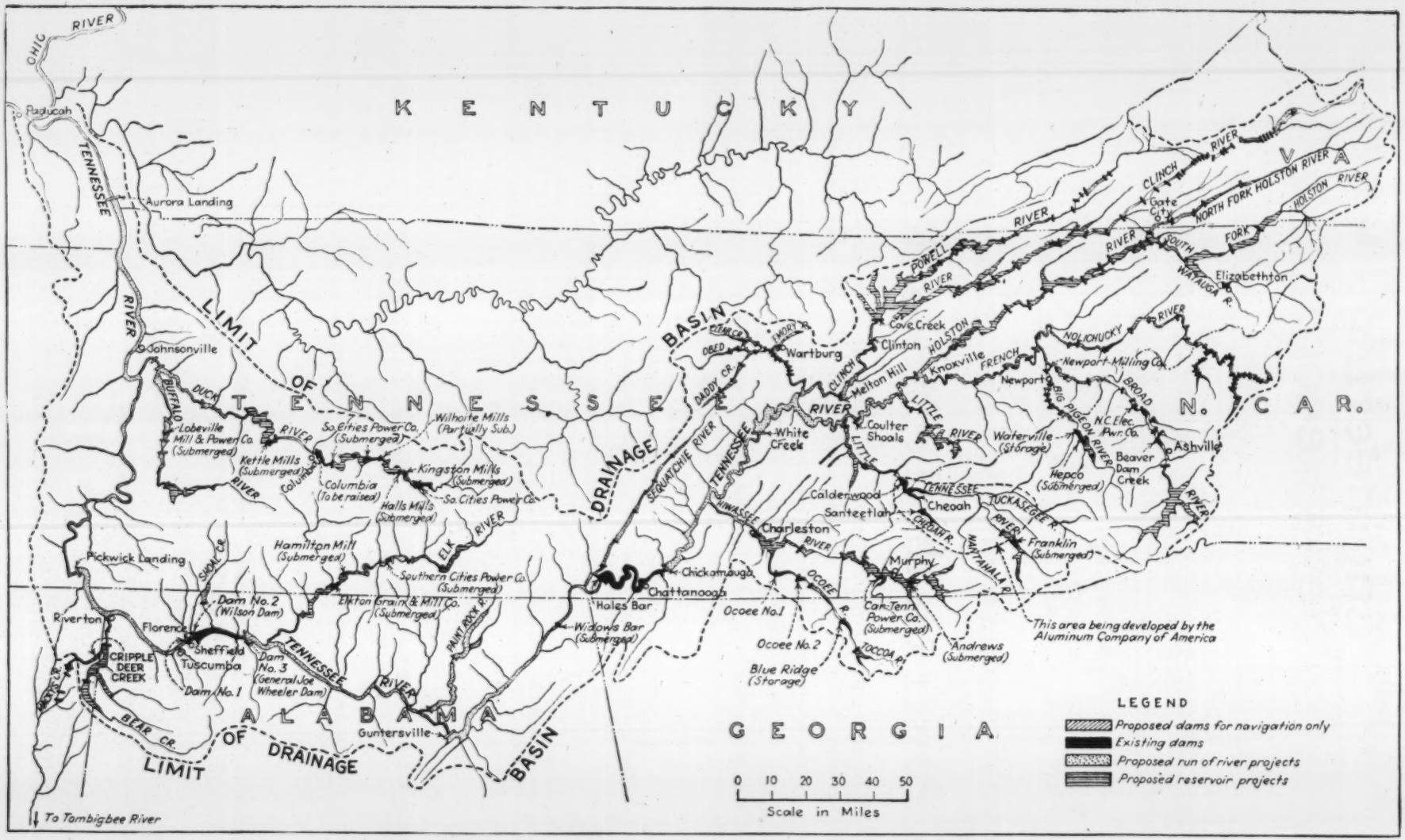
The power study of the army engineers approached the subject from two angles, first the construction of a few key plants, which with the Muscle Shoals plants would form the nucleus of a power system for the valley; and second, the development of all possible sites on the main river and its tributaries. Flood-control studies were integrated with those for power and for navigation, and the navigation studies were extended up all tributaries where there appeared any justification for considering it.

Navigation facilities

The Tennessee River is 652 miles long, has a fall of about 500 ft. and drains an area of 40,600 square miles.



Full realization of the potentialities of the Muscle Shoals hydro-electric plant waits on the construction of storage reservoirs on the tributaries, one of which is now proposed.



POWER AND NAVIGATION DAMS AND REGULATING RESERVOIRS ON THE TENNESSEE RIVER SYSTEM

The complete system of power and navigation dams and regulating reservoirs for the Tennessee River and its tributaries as developed by the Corps of Engineers is here shown. Under present conditions the river is navigable for shallow-draft barges to its head at Knoxville. This plan calls for improving the main river and extending navigation up the major tributaries, including development of Cripple Deer Creek in order to provide a connection to the Tombigbee River.

It is classed as a navigable waterway for its entire length, Paducah to Knoxville. Existing navigation developments include a lateral canal at Colbert Shoals, locks at Dam No. 1 and Dam No. 2 (Wilson Dam) at Muscle Shoals, a dam and lock at Widow's Bar, and a dam and lock at Hale's Bar. The latter was built largely by private interests. A lock at the site of Dam No. 3, above Wilson Dam, is now under construction. These facilities provide navigable depth varying from 4½ ft. at Widow's Bar to 9½ ft. at Muscle Shoals. As open-channel work has not been completed, the controlling depths vary from as little as 1 ft. in some places to 4 ft. in the section of the river between Muscle Shoals and the mouth. The total expenditure by the federal government to date amounts to $23,000,000, exclusive of the $37,000,000 charged to power production of Muscle Shoals.

Existing power development

At the time of the preparation of the district engineer's report (completed March 30, 1929) there were 23 hydro-electric power developments in the Tennessee River basin, with a total installed capacity of 413,000 kw. and plants under construction on the tributaries with a projected capacity of about 200,000 kw. In addition to the plants of a capacity of 370 kw. or more, there are twenty smaller hydro-electric plants with a total installed capacity of about 3,000 kw., and 130 water-wheel plants with a total capacity of 2,200 hp. The total installed capacity of the steam-electric plants within the basin is 180,000 kw.

The government hydro-electric plant as Muscle Shoals has an installed capacity of 184,000 kw., with provision in the power house for a total of 444,000 kw. For maximum development of the site the district engineer estimates that the total installed capacity should be 661,500 kw. The steam station at Muscle Shoals has an installed capacity of 56,500 kw. and space for an additional unit of 26,700 kw.

Floods and storage

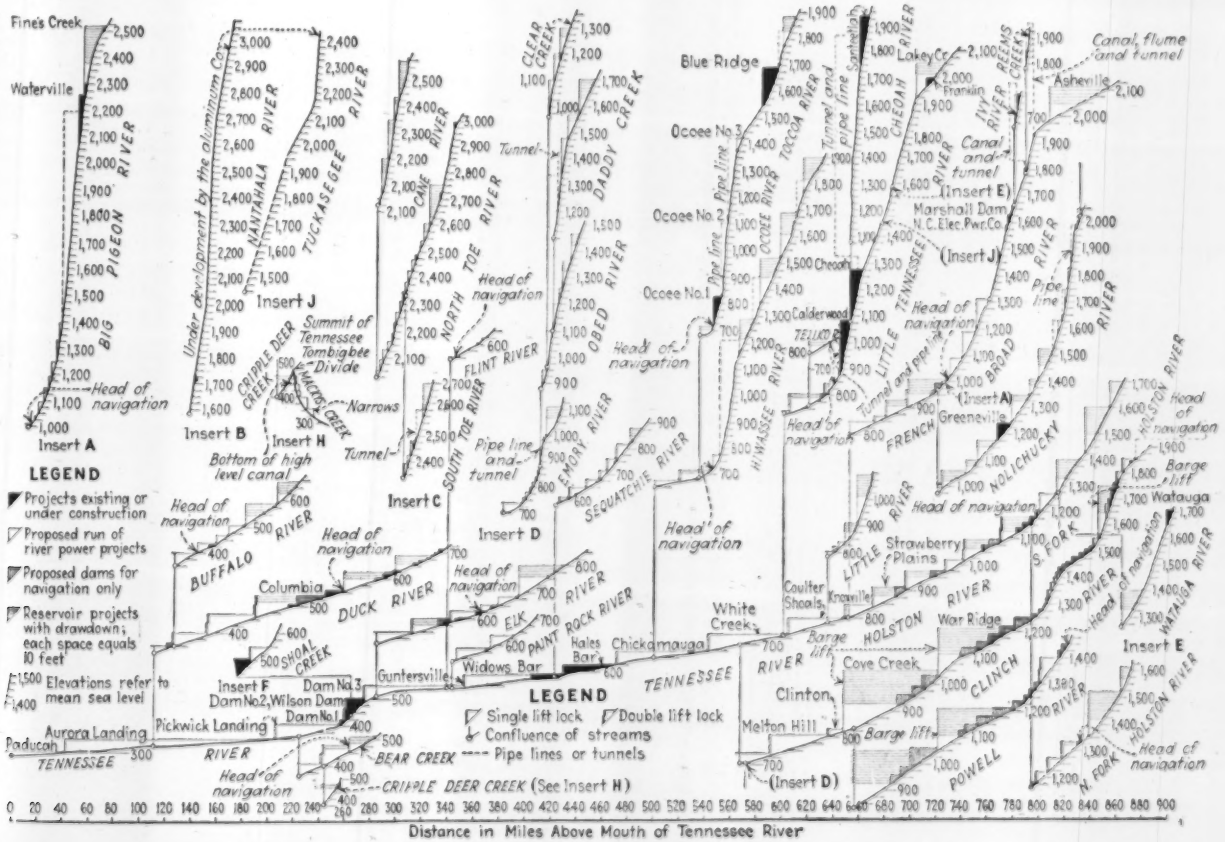
The flow of the Tennesee River varies widely. At Florence, Ala., the recorded flows range from a maximum of 455,000 sec.-ft. to a minimum flow of 4,300 sec.-ft. As a consequence, much of the installed capacity of the plants on the main river is idle a considerable part of the time. Lieut.-Col. Max C. Tyler, in a report on power at Muscle Shoals, prepared in 1925, gives the minimum flow there for the period of 1895-1923 as 8,400 sec.-ft. except for the year 1904, when the flow is variously estimated from 6,000 to 7,000 sec.-ft. Construction of the Cove Creek storage reservoir, Col. Tyler estimated, would increase the low flow to 18,000 sec.-ft. for a dry year and 20,000 sec.-ft. for an average year.

The only usable storage within the basin at the present time is 156,000 acre-ft., of which 115,000 acre-ft. is at the Santeetlah plant of the Aluminum Company of America on the Cheoah River. Additional plants contemplated by this company and the Tennessee Electric Power Co., under licenses they now hold, will add 483,000 acre-ft. of storage.

The useful storage in all existing reservoirs and in the reservoirs proposed in the army engineers' plan for complete development of the river and its tributaries is estimated at 15,760,000 acre-ft. In addition, the engineers' plan calls for a flood-control surcharge pondage 5 ft. in depth on the reservoirs of all projects on the tributaries and a surcharge of 10 ft. on pools behind dams on the main river. About half the surcharge pondage of the main river may be used for storage during the period April 15 to Dec. 15. The natural flood storage of the main river from Knoxville to the mouth is very large, estimated at 14,600,000 acre-ft.

Flood control

The study of the army engineers indicates that the use of the storage reservoirs for the regulation of the stream-



Profiles of the Tennessee River and its tributaries, showing the power and navigation dams proposed by the army engineers. On all streams where navigation is provided for, there are to be navigation dams upstream from the storage dams, as at Cove Creek, to provide navigable channels between the high and low level of the storage reservoirs. These dams and their locks will be submerged when reservoirs are full.

TABLE I — TENNESSEE RIVER DEVELOPMENT FOR NAVIGATION AND POWER

Dams, Existing or Proposed	Miles Above Mouth	Lock	Lift in Feet	Hydro-Electric Installation in Kilowatts	Cost of Navigation Facilities	Cost of Work Useful for Both Power and Navigation	Cost of Power Houses and Equipment	Total for Power
Aurora Landing	43	Single	50	333,600	$2,115,000	24,711,000	23,051,000	49,87
Pickwick Landing	206	Single	56	342,500	3,704,000	17,535,000	24,321,000	45,56
Dam No. 1*	258	Single	8	.....	.....	.....	.....	.....
Dam No. 2, Wilson Dam*	259	Double	97.1	(184,000)† 573,500	.....	.....	.....	.....
Dam No. 3	275	Single	49.5	352,920	1,709,000	17,281,000	25,860,000	44,85
Guntersville	354	Single	38	210,690	2,161,000	10,219,000	19,277,000	31,65
Hales Bar*	431	Single	40	161,300	.....	.....	.....	.....
Chickamauga	472	Single	47	216,700	3,094,000	11,633,000	16,109,000	30,83
White Creek	543	Single	64	246,390	2,755,000	9,134,000	17,388,000	29,27
Coulter Shoals	605	Single	62.5	122,365	3,363,000	4,772,000	8,734,000	16,86
Total				2,559,965	18,901,000	95,285,000	134,740,000	248,92

* Existing plants. † Present installation.

flow for navigation and power, together with the surcharge pondage behind the dams, will provide satisfactory flood control for the main river and to a lesser extent for the tributaries; it indicates further that the use of reservoirs primarily for flood control is inadvisable. Without the use of surcharge at the dams on the main stream, all reservoirs of the entire system would not reduce the height of floods on the lower rivers. The district engineer estimates that if in 1926 the entire project had been constructed and operated as planned, it would have reduced the height of the flood of that year by an amount varying from 8 ft. at Knoxville at the head of the river to 12.8 ft. at Johnsonville near its mouth. The effect at Chattanooga, where the greatest concentration of flood damage occurred, would have been to lower the flood crest by 20.8 ft. Such a reduction in flood height would largely have prevented the damage caused by the 1926 flood.

Storage reservoirs on the Tennessee and its tributaries, even if operated primarily for flood control, would have no marked effect on floods in the Mississippi River.

Ultimate river development

As required by law, the district engineer estimated the cost of developing the entire river system for power, flood control and navigation. The total amounted to $1,200,000,000 and included works on the main river to provide a navigable depth of 9 ft. from the mouth to Knoxville and from 9 to 6 ft. for a considerable distance up all the major tributaries. The power system included 149 hydro-electric plants together with auxiliary steam stations as well as storage reservoirs. In this analysis it was assumed that the entire cost of the combined system would be charged to power, except the cost of locks, barge lifts and channel work required solely for navigation. Under these assumptions the system would produce about 3,000,000 kw. of firm power under a 50 per cent load factor at an average cost of 4⅛ mills per kilowatt-hour at the bus-bars of the station.

The structure proposed for Dam No. 3 will be similiar to Dam No. 2 at Muscle Shoals except that it will have considerable less height.

Partial development

The district engineer recommended that for the immediate navigation and power needs of the region only seven dams be built on the main river. With the existing dam at Muscle Shoals and Hales Bar and with Dam No. 52 on the Ohio, these dams would provide 9-ft. navigation from the mouth to Knoxville. The total cost of the new work, including the power plants, was estimated to be $249,000,000. Eliminating power from consideration, the district engineer estimated that 9-ft. navigation to Knoxville could be provided at a cost of $56,000,000 to $75,000,000 depending on the size of the locks used. On the basis of rail-freight movement in the year 1926,

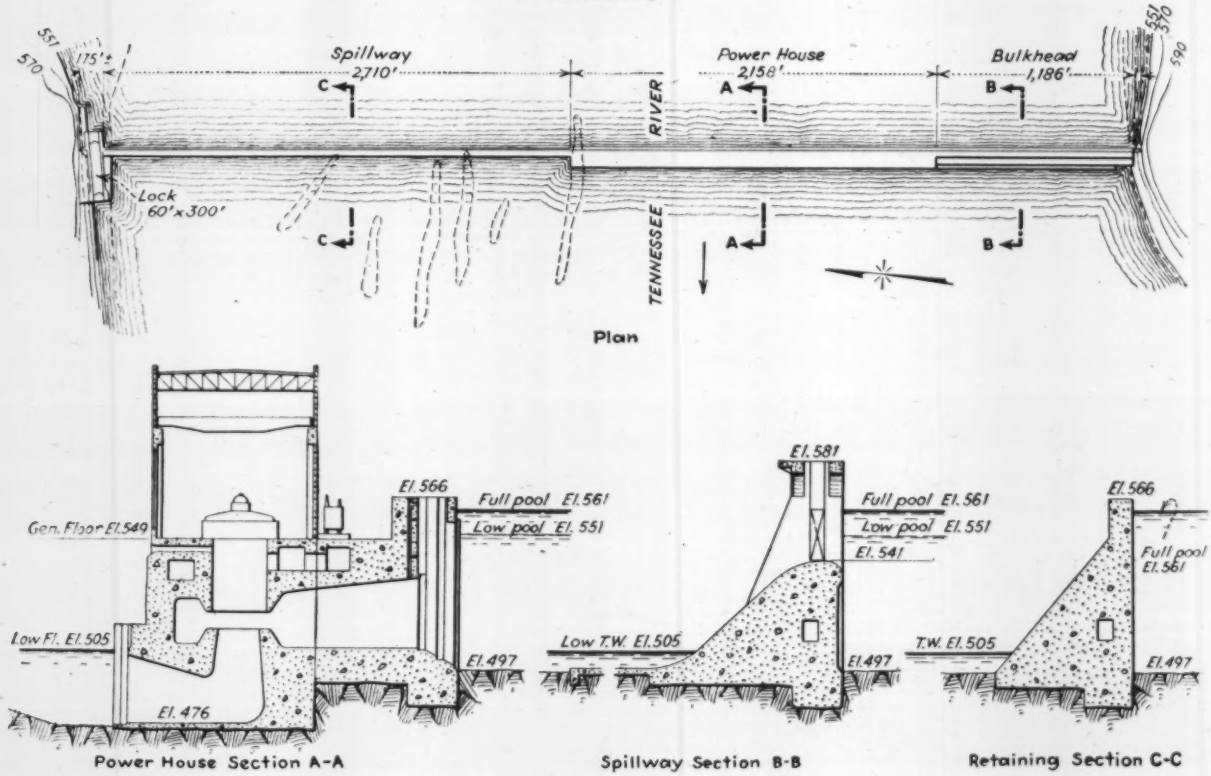


TABLE II — COST AND POWER INSTALLATIONS OF EXISTING AND PROPOSED PROJECTS

Project	Head in Feet	Hydro-Electric Installation, Kilowatts	—Useful Storage— Reservoir, Acre-Ft.	Surcharge, Acre-Ft.	Average Flow, Sec.-Ft.	Cost of Navigation Facilities	Cost of Dam	Cost of Reservoir	Cost of Power House and Equipment	Total Cost Exclusive at Navigation	Cost Per Kw
Dam No. 2 (Wilson Dam)[a]	97.1	184,000[b]	.....	.....	53,300	$9,000,000[c]	.....	.....	$37,000,000	$37,000,000	$201
		260,000	.....	.....	53,300	.....	.....	.....	8,000,000[d]	8,000,000	102[e]
Dam No. 3	49.5	352,920	182,500	365,000	51,300	1,709,000	11,597,000	5,683,000	25,860,000	43,140,000	122
Hales Bar[a]	40.	161,300	.....	.....	38,500	.....	.....	.....	.....	29,308,000	175
Cove Creek	220	165,000	3,985,830	387,000	4,600	.....	12,705,000	15,252,000	6,033,000	33,990,000	205[f]
Total	....	1,123,220	.....	.....	.....	.....	.....	20,935,000	76,893,000	$151,438,000	
New work		777,920				$1,709,000	24,302,000	20,935,000	39,893,000	85,130,000	110

[a] Existing dam.
[b] Present installation.
[c] Includes part of dam and reservoir costs.
[d] From an earlier report.
[e] Army engineer report gives a unit cost of $95 for a total installation of 661,500 kw. and a cost of $63,018,000 exclusive of navigation.
[f] Army engineer report gives a unit cost of $304 based on same total cost but an installation of 111,700 kw.

these navigation facilities would produce an annual saving of $7,300,000. The board of engineers for rivers and harbors, in its comment on the report, stated that the saving would be much less than that estimated by the district engineer.

Details of the cost of the seven dams, locks and power plants proposed for this initial development are given herewith in Table I.

Preliminary power-system studies

Approaching the development plans from the power angle, the district engineer made a study of the cost of producing power in a system consisting of the Cove Creek, Clinton, Melton Hill and Kingston plants on the Clinch River and the Hales Bar, Dam No. 3 and Dam No. 2 plants on the Tennessee River This system with steam auxiliaries, he estimated, could produce 692,800 kw. of firm power at a 50 per cent load factor. The power would cost 3.51 mills per kilowatt-hour at the bus-bar. The system here outlined is substantially that contemplated in the offer of the American Cyanamid Co. for the nitrate and power properties at Muscle Shoals.

Proposed work

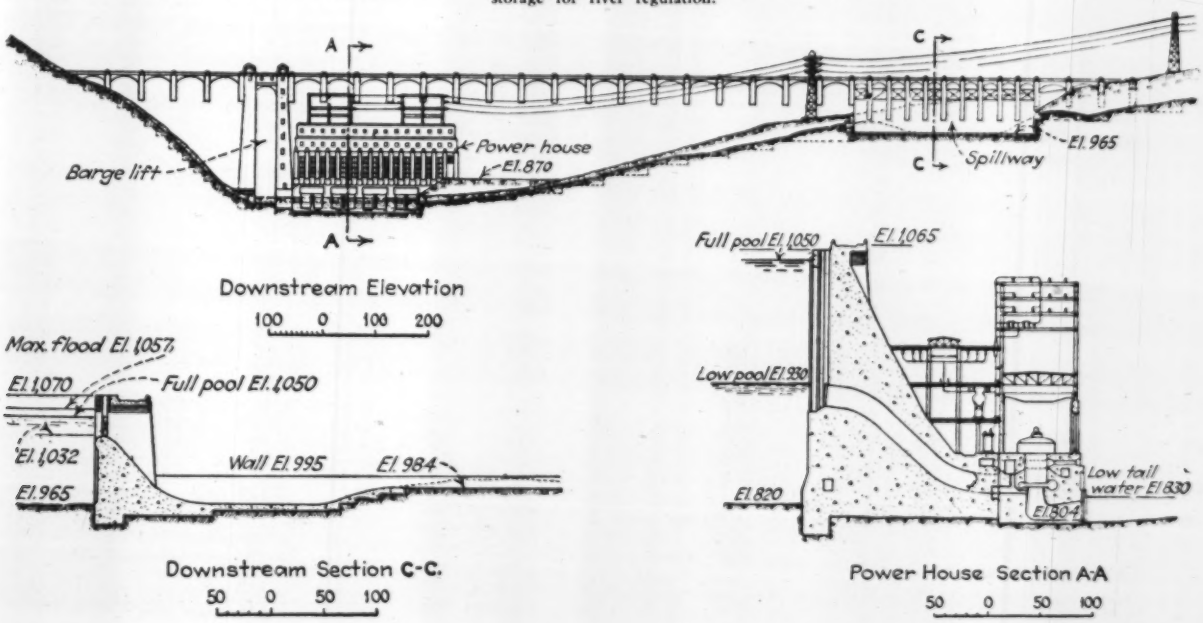
The legislation now before Congress calls for construction of the Cove Creek dam on the Clinch River, a combined power and storage project and the major source of regulation for the main river, and construction of Dam No. 3 (Gen. Joe Wheeler Dam). Under the broad powers granted to the power board it is probable that it will complete the installation of generating units in the power house at Dam No. 2 at Muscle Shoals and in the steam station there. The board also is authorized to build a transmission line from Cove Creek to Muscle Shoals if it is found that satisfactory arrangements cannot be made for use of the existing transmission lines.

The cost of the dams, reservoirs, navigation works and hydro-electric plants included in the proposed undertaking is given in Table II. To those figures should be added $1,000,000 for the addition of 26,700 kw. to the steam station at Muscle Shoals and, possibly, $4,835,000 to the transmission line, making a total of $91,000,000. The legislation now before Congress authorizes an immediate appropriation of $10,000,000 and a total expenditure of $60,000,000.

Information given in Table II is taken from the army engineer's report, except as indicated in the footnote. In the chief of engineers' summary statement, accompanying the report, it is recommended that the installation at Muscle Shoals (Dam No. 2) be increased from the present 184,000 to 444,000 kw., the capacity of the present power house; but the detailed estimates of the district engineer are based on a total capacity of 661,500 kw. and a cost, exclusive of navigation charges, of $63,018,000. However, as this larger installation does not appear justified until there is more complete regulation of the river than is provided by the partial development now contemplated, the cost of this larger installation has not been included in the tabulation.

No figure for the cost of navigation facilities has been included in the Cove Creek Dam because construction of the dams between Cove Creek and Knoxville is not contemplated in the present work. The district engineers' estimates for the Cove Creek dam includes a barge lift costing $2,550,000. Details studies for the dam may make it appear desirable to build some part of the lift chamber to facilitate completing it with the reservoir in operation, in which case the cost will be higher than that shown in Table II.

The Cove Creek dam is designed to serve both the requirements of power and storage for river regulation.



Testing Riveted Joints of Cromansil Steel

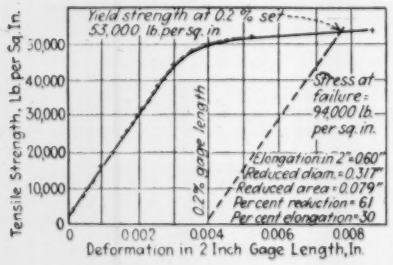
By Inge Lyse

Research Assistant Professor of Engineering Materials, Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pa.

A RECENT investigation of riveted joints provides some of the first information available on the relatively new cromansil steel. This is a low-alloy steel developed for structural purposes containing 0.40 to 0.60 chromium, 1.10 to 1.40 manganese, 0.70 to 0.80 silicon, and carbon to suit. A carbon of 0.20 provides a steel workable in the "as rolled" state, with yield-point strength of about 56,000, ultimate tensile strength of about 87,000 and shearing strength of about 65,000 lb. per sq.in. A typical stress-strain diagram is shown in Fig. 1 for steel produced by the Lukens Steel Co.

Two butt joints and two lap joints were tested, using both cromansil steel plates and cromansil steel rivets. One of each kind of joint was riveted by means of an air hammer, and the other one by means of hydraulic pressure. The plates were $\frac{5}{8}$ in. thick and 12 in. wide. Three rows of 1-in. rivets were used in all joints. Thus the butt joints contained eighteen rivets stressed in double shear and the lap joints nine rivets stressed in single shear.

Fig. 1—Typical stress-strain diagram of cromansil steel reveals its special properties.


Yield strength at 0.2 % set
55,000 lb per sq. in.
Stress at failure = 94,000 lb. per sq. in.
0.2% gage length
% Elongation in 2" = 060"
Reduced diam. = 0.317"
Reduced area = 0.079"
Per cent reduction = 61
Per cent elongation = 30
Tensile Strength, Lb per Sq. In.
Deformation in 2 Inch Gage Length, In.

Both strains and slip were measured. A summary of the strength results is given in the accompanying table. The difference in method of riveting is shown to have a marked effect upon the load at which the first slip occurred. For the butt joint the hydraulic riveting produced more than $2\frac{1}{2}$ times greater stress at time of slip than did air-hammer riveting. For the lap joint the corresponding value was about $1\frac{1}{4}$. Since the butt joint failed in the main plate, the ultimate strength was not affected by the method of riveting. The lap joint, however, showed an increase in strength of about 18 per cent by the use of hydraulic pressure, and the failure occurred through the rivets. Since the first slip in a joint probably occurs when the load becomes greater than the friction between the plates, it is reasonable that the stress at first slip was greater for hydraulic riveting where a high pressure was used than for air-hammer riveting. The stress at which the plate in the butt joints failed was about 106,000 lb. per sq.in. when based on net plate section. The crushing stress at failure was 104,000 lb. per sq.in., or nearly as great as the ultimate tensile stress. The lap joints developed

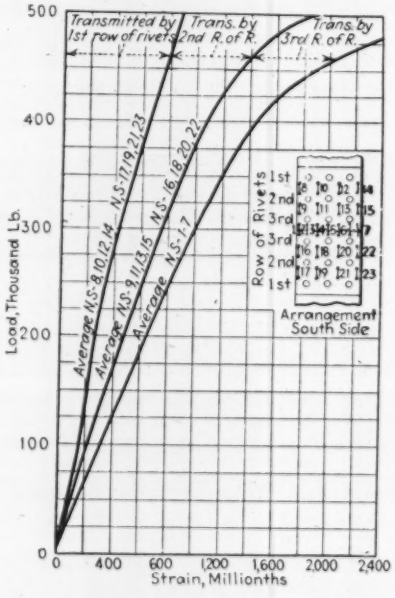


Fig. 2—Distribution of stress in cover-plates of butt joint shows a fairly uniform division of load between the three rows of rivets.

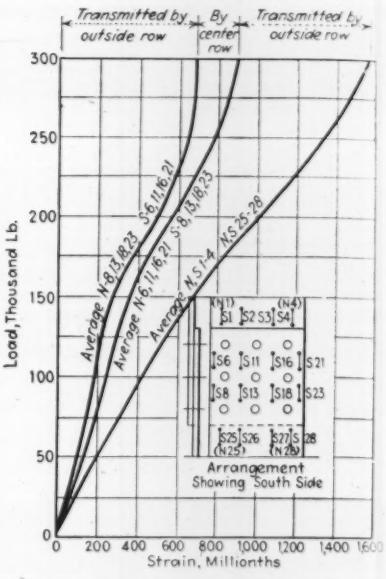
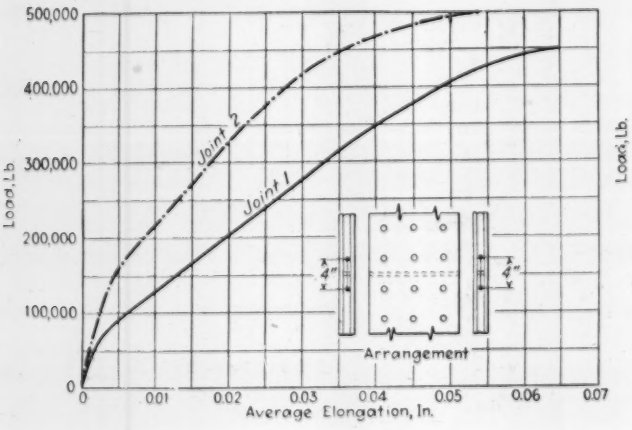


Fig. 3—Distribution of stress in the plates of one of the lap joints. Strains given are per unit of plate area.



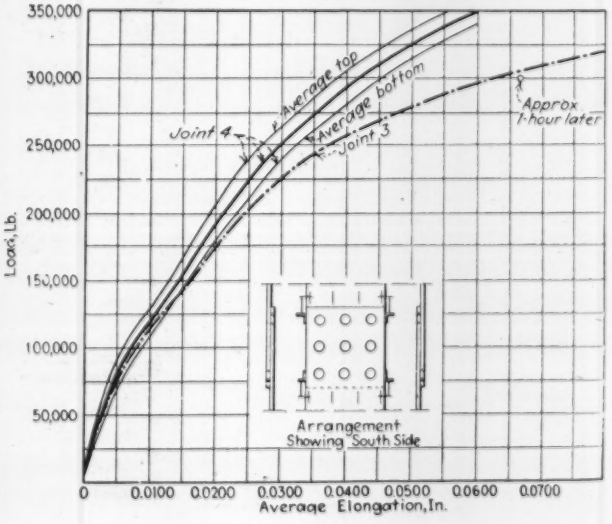


Fig. 4—Slip of plates in the riveted joints. On butt joints (left) slip was determined by measuring the opening of the joint, and on lap joints (right) by observing the average elongation of the ends of the plates. The breaks in the curves indicate the points where friction between the plates was exceeded by the applied load.

TABLE I — TEST RESULTS OF RIVETED JOINTS OF CROMANSIL STEEL

Joint No.	Type of Joint	Ultimate Load Lb.	Type of Failure	Stress in Plate* at Failure, Lb. per Sq.In.	Stress in Rivets at Failure, Lb. per Sq.In.	Load at First Slip (Approx.) Lb.	Stress in Plate* at First Slip (Approx.) Lb. per Sq.In.
1	Butt, air	583,700	In plate	106,000	103,800† / 41,200‡	60,000	11,200
2	Butt, hydr.	585,500	In plate	106,500	104,000† / 41,400‡	155,000	28,200
3	Lap, air	456,000	Through rivets	83,000	67,000§	75,000	13,600
4	Lap, hydr.	533,000	Through rivets	97,000	78,200§	90,000	16,400

* On net area (55 sq.in.) † In bearing. ‡ In double shear. § In single shear.

ultimate stress in single shear of 67,000 lb. per sq.in. for air hammer and 78,200 lb. per sq.in. for hydraulic riveting, indicating that a very high-grade rivet material was used.

The distribution of the stresses in the coverplates of the butt joints is illustrated in Fig. 2. All three rows of rivets carried load from the very beginning of the test; and while the first and second rows carried more than did the third row, the difference was relatively small. At higher total loads the amount carried by each row was very nearly the same. The stress distribution in the plates of the lap joints is illustrated in Fig. 3. In the lap joint the center row of rivets seemed to transmit a smaller load than did the two other rows. However, when the rivets in the latter rows had been stressed beyond their yield point in shear, the center row had to take a greater portion of the load, so that at the ultimate load all rows were probably carrying approximately the same amount. Thus the ordinary method of dividing the total load on the joint by the number of rivets probably gives nearly correct stress distribution at time of failure.

The slip of the plates in both butt and lap joints is illustrated in Fig. 4. The load at which the friction between the plates is exceeded is indicated by the break in the curves. The rigidity of the joint is much greater when the friction is carrying the load than when the rivets are stressed in shear. Thus the rigidity of a riveted joint is probably reduced appreciably when working stresses become in excess of the stresses that cause slip of the plates.

Flexible Lining of Tile Used for Drainage Ditch

CEDAR FORK, a stream that flows diagonally through the city of Galesburg, Ill., has been paved with a novel flexible lining of vitrified-tile blocks for some 3½ miles. The stream carries the runoff of a 10-square-mile drainage area as well as sanitary and storm sewage for practically the entire city. Drainage from floods has been relatively frequent and at times serious. During dry periods the only flow coming from the sewers resulted in a foul condition, and property values were affected. A parallel intercepter to care for sanitary flow, and relocation, enlargement and partial paving of Cedar Fork Creek were decided upon as a solution to the problem. Construction during the last 2½ years involving $1,250,000 has recently been completed by the Galesburg Sanitary District. The new channel cross-section, with a bottom width varying from 10 to 40 ft., has 1½-to-1 side slopes except for those portions running through congested districts, where a concrete channel with vertical side walls was adopted to economize on space.

The liner blocks, made by a local paving-brick company, are 12x12 in. by 3 in. thick. Two edges are respectively tongue and groove. Two ¼-in. holes parallel to these edges run through the block. In laying the blocks they were staggered in such a manner that ½-in. steel bars in these holes fastened each block to two adjacent blocks on either end. A flexible mat is thus obtained, and its tendency to slide is reduced by drainage at 12-in. intervals. It is not affected by freezing and thawing and is easily repaired through replacement of individual blocks if this should become necessary.

During construction a flood of 10 ft. per second furnished a test flow, undercutting and scouring the exposed downstream edges. The paving merely turned down or doubled over, and repairs were nominal in cost. By spraying twice with chemicals last summer the weed growth through the joints was checked. Some of the paving has passed successfully its second winter.

Alvord, Burdick and Howson, consulting engineers, Chicago, made the preliminary report, prepared the detailed plans and supervised the construction of the lining for the Galesburg Sanitary District.

Fig. 1—Section of Cedar Fork Creek paved with vitrified tile held together by steel rods running through the tiles.

Newspaper Building Floor Designed for 2,100-Lb. Load

The new Times-Star Building in Cincinnati, supported with pipe-pile foundations, has its seventeen-story office section insulated from six-story printing section, on one floor of which a 2,100-lb. live load is provided for

By W. A. Lensky
Structural Engineer, Samuel Hannaford & Sons, Architects, Cincinnati, Ohio

A PRESS-ROOM FLOOR, designed for a 2,100-lb. live load, and a six-story publication section insulated against the transmission of sound and vibration to a seventeen-story office section are conspicuous structural features of the new Times-Star Building, at Cincinnati, Ohio. Its site, at Broadway and 8th St., has a frontage of about 136 ft. on Broadway and extends back on 8th St. to Eggleston Ave. The structure consists of two sections; the front portion, 136x80 ft., is a seventeen-story steel-frame tower, while the rear six-story portion, 136x200 ft., is of reinforced concrete with floors of the flat-slab type. A basement and sub-basement extend under the entire building; but since the ground slopes down from Broadway, the rear of the basement is above grade at Eggleston Ave. There is a one-story extension, 49x75 ft., providing an extra service room in connection with the garage, which is in the basement.

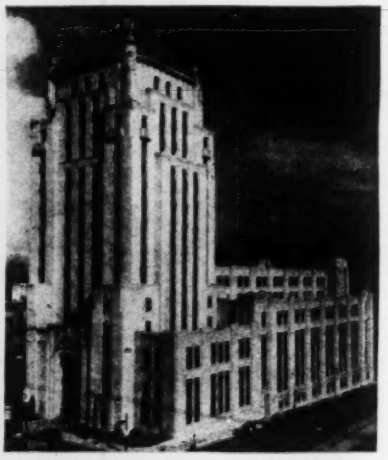


Fig. 1—Times-Star Building at Cincinnati has its seventeen-story steel frame office section insulated against noise and vibration from its concrete six-story publication section in the rear.

The main shaft of the tower starts at the seventh floor and is about 77 ft. square, reduced by numerous offsets as the tower rises to its height of 235 ft. above the sidewalk on Broadway. The rear, or manufacturing section, has one bay offset at the fifth floor on the north and south elevations. In Fig. 1 is a view of the building with its finished limestone facing.

Although the tower and manufacturing sections are inter-connected the walls separating the reel and press rooms from the tower section are insulated against sound transmission; and wherever the floors of the manufacturing section adjoin the tower section they are separated with a ½-in. expansion joint. This separation is carried through the entire building, including walls and foundations, so as to take account of the difference in height, type of construction, possible transmission of vibrations from the manufacturing to the tower portion, and the very remote possibility of unequal settlement of the two sections. In effect, the building, although a unit, is really made up of two buildings of different types of construction.

Pipe-pile foundations

The site is located directly over an old canal, now filled. Borings carried to rock disclosed a layer of about 20 ft. of filled ground, below which was a layer of sand and gravel (under the tower portion), followed by soft blue clay to rock over the entire site. The bedrock consisted of hard limestone about 110 ft. below Broadway, overlain with softer layers of blue shale. Excavation for the basement and sub-basement was carried out in the open. Steel sheetpiling to brace the banks was driven soon after the excavation of the center portion started, and earth banks were left against the sheetpiling until the first tier of interior footings was installed. Then the banks were removed and permanent diagonal bracing from the footings to the sheetpiling was installed.

Pile foundations for both sections of the building consist of steel tubes driven to rock, filled with concrete and capped with reinforced concrete. Work was started, however, with composite piles, having a lower portion of 10-in. steel pipe ⅜ in. thick, averaging 53½ ft. long, driven to rock and filled with concrete. The upper portion was a 16-in. uncased concrete cylindrical pile averaging 20 ft. 8 in. in length, enveloping the steel pipe for 18 in. to form a splice. After installing a small number of these piles under the low portion of the building, it was found that the progress of the work was much slower than expected: the resistance to driving was very great, due to consolidation of soil in the pile cluster, which caused considerable damage to driving equipment. In addition, the ratio of driven pipe length to

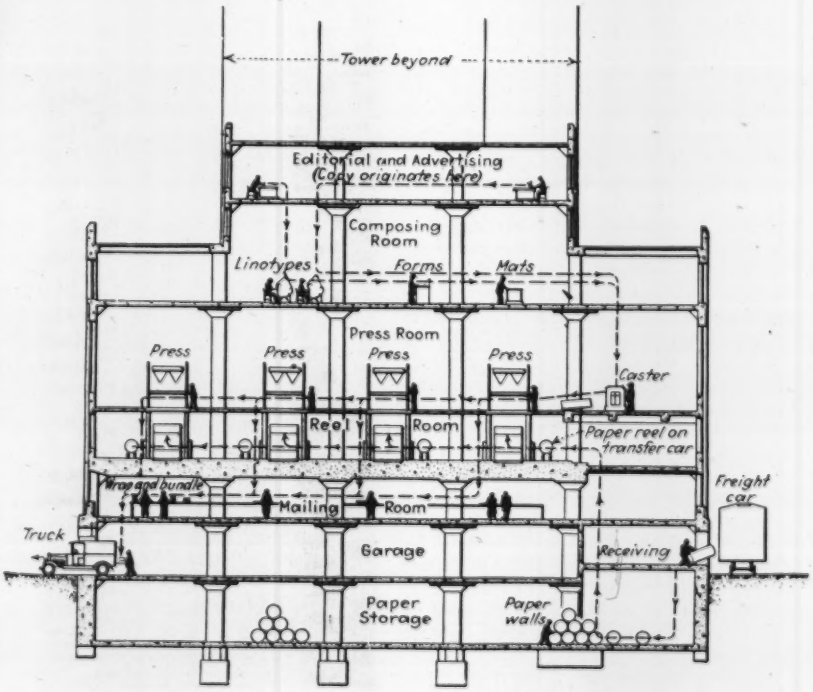


Fig. 2—In the six-story reinforced-concrete manufacturing section of the Times-Star Building the heavy floor under the presses is particularly notable.

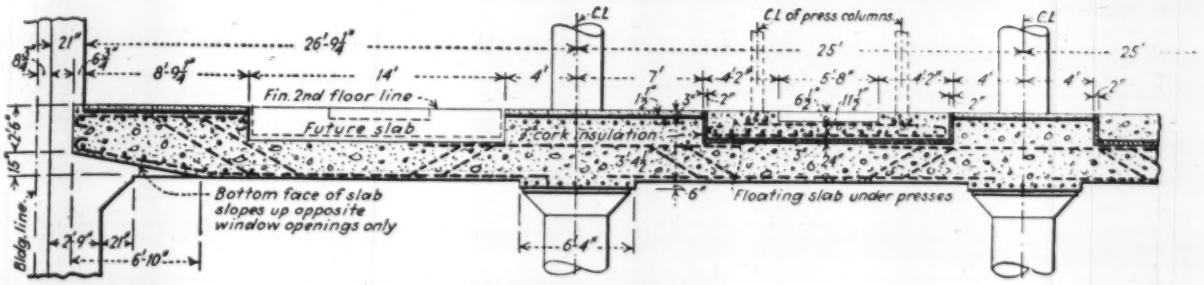


Fig. 3—Cross-section of floor carrying printing presses and designed for a 2,100-lb. live load. Note "floating slab" construction that insulates press vibrations from structural slab.

its diameter exceeded generally accepted recommendations and code requirements.

It was then decided to change the type of piles to 24-in. steel pipes. Specifications for this type of pile were prepared to conform to the requirements of the New York building code. The carrying capacity of steel pipe was assumed at 7,500 lb. per sq.in. on the area of steel shell (after deducting ⅛ in. thickness for possible rusting) and 500 lb. per sq.in. on the contained concrete area. Each 24-in. pile was seated in the rock by drilling from 2 to 4 ft. deep to assure absolute bearing. The thickness of pipes varied from ⅜ to ½ in., depending on loading. They were made of seamless steel tubing in lengths of about 25 ft.

In general, not more than three lengths were used for any one pile. The lengths were joined together by welding on the outside and inside, without the use of external or internal couplings. Each pipe was inspected after sinking, to discover possible damages to the pipe and the welds during driving. In only one or two cases were the welds fractured, throwing the sections of pipe out of alignment. This difficulty was remedied by inserting a long internal pipe sleeve at the fracture, the outside diameter of this sleeve being slightly under the inside diameter of the pipe, and the lower end of the sleeve being tapered. After driving and centering this sleeve on the joint, both sections of pipe were brought again to almost perfect vertical alignment.

For sinking the piles a smaller pipe, called the core, was inserted in the 24-in. pipe, and both pipes were driven simultaneously. The ends of both pipes were open. At intervals the core was withdrawn, and its contents were dumped out by opening a trap arrangement at the bottom. The clay was of such a soft and sticky nature that from 10 to 15 ft. of it was lifted at a time. Very little jetting was done; all of it was on the outside of the 24-in. pipe, and was done only to expedite penetration through a thin layer of compacted sand and gravel and tough clay.

All the piles were accepted as being plumb and satisfactory. Of the piles driven, only three were as much as an inch out of plumb, and as the declination from the vertical of the lower end was toward the outside of the cluster the piles were approved. It was clearly demonstrated that the heavier pipe is preferable, the ½-in. thickness giving the best results. The cost of 24-in. pipe pile in place was about $9 per linear foot.

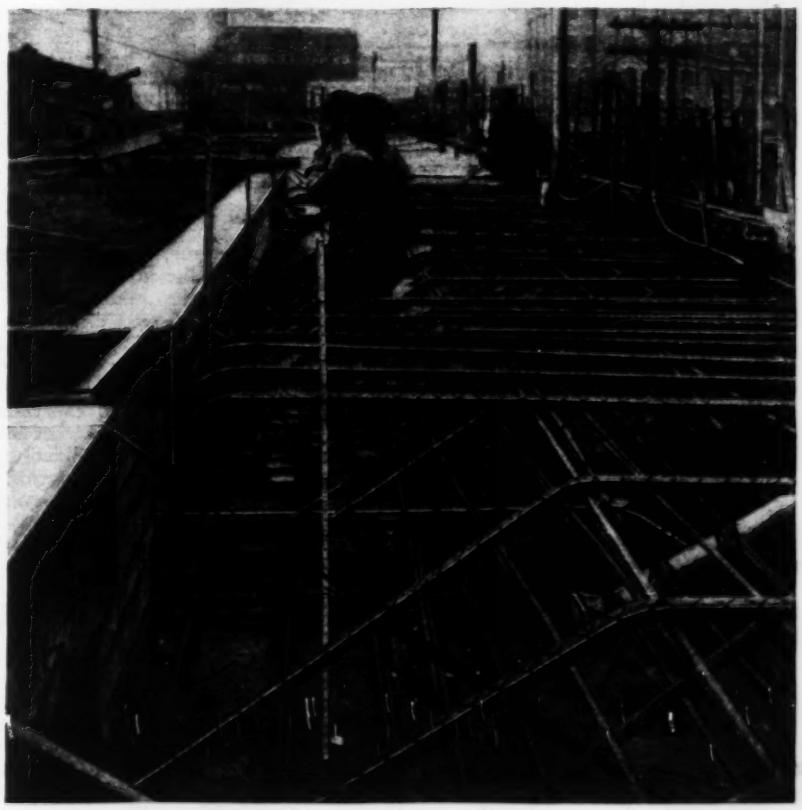
Steel framing

In the tower section the typical floor framing is mainly of heavy wide-flange I-beams and girders forming large and irregular panels for the support of a slab-and-joist type of floor. Joists 30 in. c. to c. vary from 8 to 12 in. in depth, plus 2½ in. of slab. Beams and girders are generally framed 2 in. below the top of the structural slab. Removable wood forms were used. Live loads assumed in the design were 225 lb. per sq.ft. for the sub-basement floor or paper-storage floor, 150 lb. for the basement or garage floor, 100 lb. for the first floor and 50 lb. for the remaining tower floors, assigned as ordinary office space.

Numerous architectural offsets, with limited allowance for depth of floor members and deep window reveals, imposed a problem in the design of shallow plate girders to support the offset columns. Out of 34 main columns in the tower section of the building, only eight interior columns are without offsets. The heaviest concentration of load occurs over the main entrance arch where a double-plate girder, with a span of 39½ ft., carries a building column located about 14 ft. 6 in. from one end, with a load of 687,000 lb.

All columns are of H-section, figured for a maximum allowable stress of 15,000 lb. for direct axial loading. For columns taking wind loads in addition to axial loads, stresses were increased 25 per cent. Wind load was assumed

Fig. 4—Heavy reinforcements required in reel-room floor to carry 2,100-lb. live load.



at 15 lb. per sq.ft. of vertical projection of walls exposed to the wind, and resisted by exterior column bents only. Windbracing was figured by the cantilever method, and consisted principally of top and bottom T-sections cut either from 24-in., 110-lb., wide-flange I-beams, or 15-in., 55-lb. channels, $\frac{7}{8}$-in. rivets being used. Windbracing in corner panels of the main shaft of the tower spandrels consisted of plate girders and heavy gusset plates with connection angles. All structural floor members and columns at elevator shafts are inclosed with a minimum of 2 in. of concrete; the other columns have a minimum of 4 in. of masonry.

Reinforced-concrete construction

In the manufacturing section of the building each floor is specially designed for its particular service in the publication of the paper. Fig. 2 is a cross-section, showing the disposition of space and equipment. Except for the second and fifth floors, a flat-slab design, with panels about 25 ft. square, is used. The second, or reel-room, floor (Fig. 3) is believed to be the heaviest weight-bearing floor in the United States. It carries four lines of heavy presses and reels occupying a space about 100 ft. wide and 155 ft. long. This section of floor, designed for 2,100 lb. per sq.ft. live load, is supported by 42-in. round columns reinforced with spirals and spaced 25 ft. on centers. The remainder of the floor is designed for 100 to 350 lb.

The frames supporting each line of printing presses stand on "floating" or insulated reinforced-concrete slabs having a minimum thickness of 11½ in. under the transfer pits, which are 6½ in. deep. Each floating slab is 14x155 ft. and is insulated from the structural slab by 3 in. of special cork on the bottom and 2 in. on the sides. The structural slab is 24 in. thick under the floating slab and 40½ in. thick over columns and between the lines of floating slabs. The thicker portion of the slab carries 3 in. of cement finish, which is insulated from the structural slab by 1½ in. of special cork, and is designed for 150 lb. live load to carry an industrial track for moving rolls of paper. The 24-in. structural slab is designed as a simple slab framing into the 40½-in. slab over the columns, which in turn is designed as an inverted continuous rectangular beam. This slab is heavily reinforced, extra steel being used to take care of shrinkage stresses, as shown in Fig. 4.

It was specified that the reel-room floor should be poured without stop, thereby eliminating any construction or expansion joints. The intention was to obtain a floor of uniform concrete consistency, strength and rigidity, which would act as a unit in resisting the transmission of vibrations. The contractor deposited about 2,100 cu.yd. of concrete in one continuous operation under winter conditions, the average temperature for three days being 41 deg. F., with a high average of 46 deg. on Dec. 31 and a low average of 38 deg. on Dec. 29, 1931. The slab was reinforced with 176 tons of reinforcing rods, all bent in the shop and carefully placed in the forms in accordance with details. The results were very successful. Every precaution was taken to sustain the great weight of fresh concrete, and a special layout of shoring was prepared by the contractor, with shores left in place under each floor down to the sub-basement. The general appearance of the floor during the deposition of concrete and the arrangement of reinforcing steel are shown in Fig. 5.

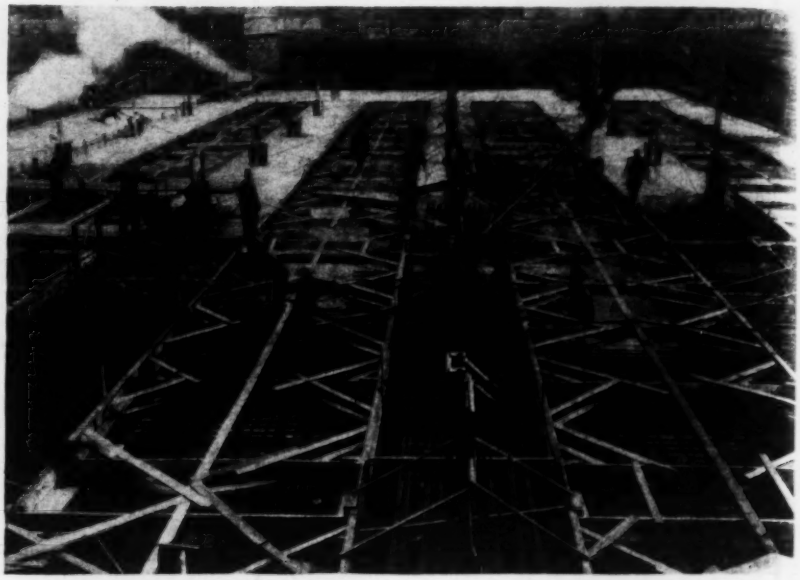


Fig. 5—Concreting the floor for the presses. Formwork shown is for floating slabs under presses, which are insulated from the structural slab by 2 in. of cork.

To facilitate rapid progress of concreting, the aggregates were first separated in proper proportion at the plant of the supply company, then mixed and hauled in trucks to the site, where they were deposited in the mixers. From the mixer the concrete was discharged into the bucket of a hoisting tower and then poured through chutes into buggies and conveyed to the point of deposit. The average slump of concrete for this floor was about 6. Electrical vibrators were used for consolidating the concrete in removable steel forms.

Panels adjoining exterior columns on the south side are supported on columns and brackets off the columns, the column width being the entire space between window openings. Opposite window openings, the bottom side of the slab directly over the openings tapers up 15 in., so as to clear the window head. Spandrel beams between these columns are practically eliminated, as the architectural features prevented the use of an inverted spandrel. It was necessary, therefore, to transmit the floor loads opposite windows to a strip of floor supported by brackets, causing considerable moment in exterior columns.

Concrete design

Concrete working stresses were based upon concrete designed to produce a minimum-standard, 28-day compressive strength of 3,000 lb. per sq.in., and 3,500 lb. for interior sub-basement columns. Reinforcing steel was designed on the basis of intermediate-grade new billet steel. As the strength and uniformity of concrete were of prime importance, the field design and control of concrete mixes for definite and predetermined strengths were carried out under the architect's specifications by an independent engineer, who furnished and set up a field laboratory for testing aggregates and various field concrete samples. In addition to field testing, which was made to check the method of curing the concrete on the job, the standard laboratory-cured test cylinders were made as a check on the field design of concrete mixes. The average unit stress of all 28-day laboratory-cured specimens was 3,930 lb. for the tower section, 4,100 lb. for the manufacturing section and 3,900 lb. for the floor under presses (reel-room).

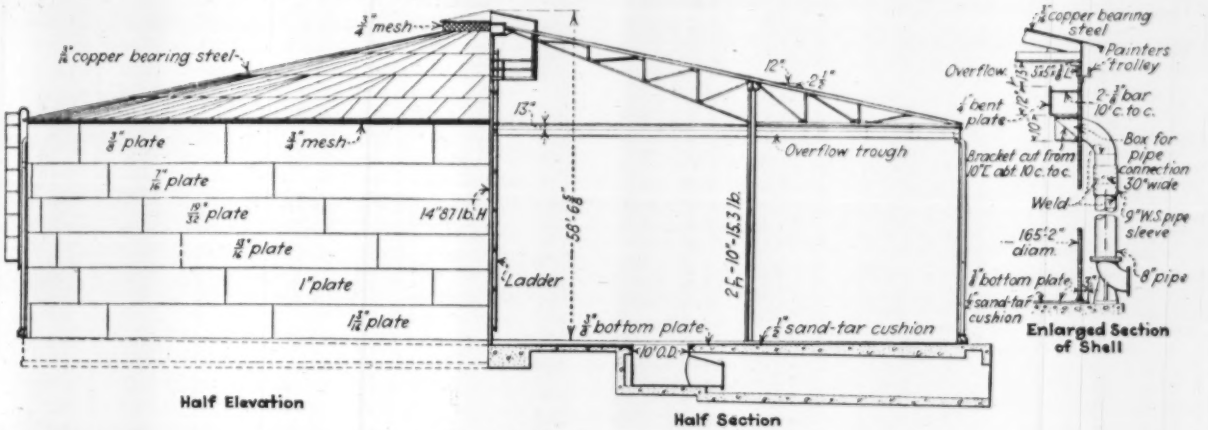
Designers and builders

Both the architectural and engineering designs were developed by Samuel Hannaford & Sons, architects, Cincinnati, Ohio, with the writer in charge of structural design. The Ferro Concrete Construction Co. was the general contractor, and the Western Foundation Co. was contractor for the piling. E. A. Hagy was engineer in charge of field design of concrete. Structural steel was erected by the Crawford-Weigel Co., Cleveland, Ohio. Ward B. Miller was the architects' construction engineer in active charge at the building.

Milwaukee Builds a Welded Tank of 6,000,000-Gal. Capacity

Said to be largest of its type yet built, the tank stores water for use in the western part of the city

CONSTRUCTION of an all-welded 6,000,000-gal. steel water tank has recently been completed by the city of Milwaukee, augmenting the storage and increasing the supply in the Menomenee Valley in the western part of the city. The tank is of the flat-bottom type and is said to be the largest welded tank yet built. The location is near the junction of a 30-in. and a 36-in. main having an outlet capacity of 100 m.g.d. The ultimate plan calls for another tank of the same capacity and a pumping station housing four 30-m.g.d. centrifugal booster pumps. The tanks will be filled at night, and the boosters will operate during the peak consumption period of the day.



The new 6,000,000-gal. water tank for the city of Milwaukee, built of welded plates, is claimed to be the largest yet constructed.

The tank is 165 ft. in diameter, 37½ ft. high to the overflow line and is covered with 3/16-in. roof plates supported on structural-steel trusses and purlins. With the exception of the structural work, all steel was welded in the field by the electric-arc process. The total weight is 700 tons.

Welding practice

Support for the tank is a 12-in. slab deposited directly on the ground and pitched toward the center. The ⅜-in. steel bottom plates comprised the first shipment to the job and were laid on a ½-in. layer of sand and asphalt, spread over the concrete slab. They were butt-welded, 3x¼-in. chill strips being used under each joint. Inasmuch as some difficulty was experienced from buckling of the bottom due to exposure to the sun, its completion was finally deferred until the roof plates were in place. The buckled plates then were cut loose, straightened and rewelded.

Six courses varying in thickness from 1 3/16 to ⅜-in. make up the cylindrical section of the tank. Each course comprises thirteen plates, 4 ft. wide, shaped to the diameter of the tank and placed in position by three crawler cranes. These cranes operated jaw grapples, which supported the plate while the initial welding was being done. The individual plates of the bottom course weigh 4 tons each. These were welded to the base plates with double fillet welds. No angle iron was used at this connection. The top and bottom edge of all plates were machined and beveled and joined to the next course above with double-bevel butt welds. All vertical seams used solid butt welds. An angle was welded to the top of the shell to act as a stiffener and as a support for the roof trusses.

During construction cage scaffolds for the welders were hung from the top edge of the plates by means of roller supports. This arrangement permitted the welder to move his cage around the tank as his work progressed. When a course was completed a crane lifted the cage to the next course.

Sixteen radial roof trusses are supported on a column at the center and the tank side. In addition, an intermediate column is placed at the midpoint of each truss. All columns have steel base plates welded directly to the floor plates. The 3/16-in. copper-bearing steel roof plates rest on purlins and rafters framed into the trusses.

At the center of the roof is a penthouse, 16 ft. in diameter and 2 ft. high, designed to permit ventilation through a heavy wire mesh screen and to give the tank a finished appearance. Additional ventilation is provided by a screen 9 in. high tack-welded above the top of the side walls and covering the ends of the roof trusses.

In erecting the roof three sheets were welded together on the ground and then placed in position with a crawler crane equipped with an 80-ft. boom and special grappler. They were then welded to the roof framing.

An overflow trough, 12x12 in. in cross-section, extends around the inside of the tank, with the lip 1 ft. from the top of the top plate. It is supported on brackets cut from 10-in. channels and welded to the top plate at 10-ft. intervals. Six 8-in. pipes lead the water from this overflow trough down the outside of the shell to a storm sewer.

The specifications call for the entire tank to be sandblasted and then given a prime coat of chromate of lead. The exterior is then to receive two coats of aluminum paint, and the interior two additional coats of chromate of lead. Structural steel was not to be sandblasted but was given the prime coat of chromate of lead in the shop, followed by two field coats. The under side of the base plate was sandblasted, primed with one coat of chromate of lead and one heavy coat of asphalt paint before being laid. An outside painter's trolley track consists of a Z-bar welded to the tank side. An inside trolley will ride on the lip of the overflow trough.

Exclusive of the concrete foundation mat the work was carried out on a cost-plus basis by the A. O. Smith Corp., Milwaukee. It is expected that the cost will not exceed $85,000. The project was rushed into construction last summer, contract being awarded for the tank nine days after the city council approved it. Ten gasoline-powered arc-welders with crews were supplied by the Harnischfeger Corp., Milwaukee. The work was carried out under the general direction of J. C. Schwada, city engineer, with Herbert H. Brown, engineer, in direct charge of design and construction of pumping stations for the city of Milwaukee.

Making the Highway Pay for Its Cost

A chart and interpretation of its application to ascertain the justifiable capital outlay per mile and the type of improvement indicated for any highway project

By Bernard E. Gray
Highway Engineer, The Asphalt Institute, New York, N. Y.

IF THE USER of the highway should pay for its improvement, the degree of improvement should be in accordance with his ability to pay. The measure of this ability to pay is the total reasonable tax paid by the user. By reasonable tax is meant one that is less in amount than the savings in travel costs effected because of the improvement of the highway, and one that it is practicable to collect. The present taxes, on the average, are undoubtedly as high as can be safely levied. The further development of highways will be greatly advanced if these realities are recognized, instead of theorizing so much on fancied indirect savings to the motorist.

The present taxes paid throughout the country by the highway user are 4c. per gal. average for gasoline tax plus an average registration fee of $13.50 per vehicle. Figuring the average mileage per vehicle per year as about 7,000 miles, with 15 miles travel per gallon of gasoline, the tax paid per car is a little less than ½c. per mile.

Basis of chart

Using this figure as the earning per vehicle per mile, the accompanying chart shows the amount that may be safely expended per mile upon improvement. The horizontal lines in the chart represent the earnings per year for a mile of highway (as shown in the left-hand column of figures) according to the daily traffic (as shown in the right-hand column of figures). Estimating that money is worth 4 per cent, and that provision for annual depreciation at 3½ per cent will rebuild the highway periodically as required, and with an allowance for maintenance of $250 per mile, the curve shows the amount as capitalized that may be expended for the construction of a mile of road. This total capital outlay per mile is shown across the bottom of the chart.

As a result of wide study by highway engineers in the evaluation of the different types of road surfaces to withstand the ravages of traffic, an approximate grouping is indicated as follows:

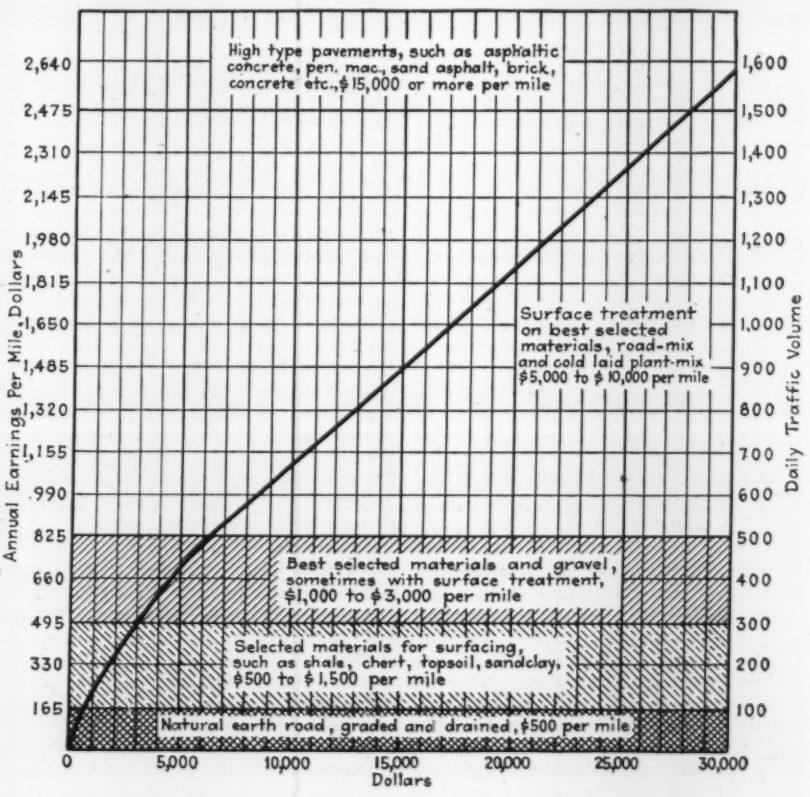
0-100 vehicles daily—Natural earth, graded and drained.
100-300 vehicles daily—Selected local materials for surfacing, such as shale, chert, top-soil, sand-clay, etc.
300-500 vehicles daily—Best selected local materials or gravel, frequently with surface treatment.
500-1,500 vehicles daily—Road-mix and cold-laid plant-mix surfaces, or heavy surface treatments.
Over 1,500 vehicles daily—Bituminous concrete, penetration, macadam, sand asphalt, concrete, brick, block, etc.

With these groups shown as five horizontal zones on the chart, the justifiable expenditure for any particular traffic is readily determined. The objection frequently raised in regard to such methods of estimating costs of construction and maintenance is that $250 per mile will not maintain various low-cost road surfaces. This is only true when the volume of traffic is greater than that for which the road was originally designed. For example, untreated surfaces under a traffic of 300 vehicles per day or less will have an annual wear less than can be replaced at this amount, plus allowance made for depreciation. The same principle applies for any of the other types. With the heavy surface treatments or road-mix surfaces, while the maintenance costs of such surfaces under a traffic of more than 2,000 vehicles per day would be over $250 per mile, yet in the range of traffic for which they are designed they can be maintained for this amount plus the allowance for depreciation that covers periodic retreatment. In the anaysis of usual maintenance figures it will be shown clearly that a major part of cost now charged to maintenance are in reality capital expenditures, because every year the road is not only as good as it was in the beginning but is decidedly better, because of widening operations, drainage improvements, guard-rail construction, surface strengthening, etc.

Stage construction

In addition to this ample provision for maintenance, still another safeguard in this method of design according to traffic is to be found in the cumulative benefits arising from stage construction. Assume the extreme case of a road improved for a traffic of 100 vehicles per day, which would call only for proper grading and drainage; then assume an increase in traffic to 300 vehicles per day; the needed cost of the further improvement would be for surfacing only, plus some widening. Again, if the traffic should increase to 600 vehicles per day, the selected materials placed previously for surfacing would become a satisfactory base for the relatively thin

Chart of justifiable capital outlay per mile for grading, draining and surfacing a mile of road based on assumed annual earnings at different daily traffic volumes.



wearing course to be placed upon it. Then finally if the traffic should reach 1,500 vehicles per day, all of this previous work could be utilized fully and the new surfacing would give a more satisfactory and economical use because of the high degree of stability in the foundation previously constructed.

Furthermore, the now increased margin of earnings from the highway over and above the amount required for its final improvement, due to this salvage of all previous work, would provide a fund for supplementing earnings on the lower-traffic-volume roads which, because of lack of local materials easily available, might require slightly increased expenditures above those that might be strictly justified on a particular mile of road. This is occasionally necessary on some routes which, for the greater part of their length, have the requisite volume of traffic to justify a given type of improvement, but which for a short mileage, either because of intersection with other routes or very sparse settlements, drop below a given earning level. This fund from surplus earings on such parts of the system enables equalization of type to be made for practical purposes

Overbuilt roads

It may not be amiss to call attention to the amount of grading that can be justified for any given highway. Grading can be of two kinds. One is where the existing roadbed is modified by flattening curves, smoothing the profile and correcting the drainage, yet generally conforming to the existing location without damage to abutting property. The other is where a new location is made without regard to property damage, with view only to perfect alignment and grade regardless of cost. The latter procedure is only justified for conditions of extremely heavy traffic, where the safety and convenience of such traffic is of paramount importance, or where property values are so small that it may be actually cheaper to select a new location than to improve the old one. Criticism, and justifiable criticism, frequently has been made by property owners of the wanton property damage caused, to say nothing of the needless expense for grading operations, when misguided design has attempted to obtain boulevard alignment for a back-country road. The defense of such design that future traffic will demand these refinements is seldom borne out, and an intelligent appraisal of local conditions should lead to a correct procedure in this respect.

Knowing the great need for further highway improvement, it will amaze many people, even many engineers, to know that much over-design has already taken place. For example, there are only about 50,000 miles of rural highways in the United States that carry more than 1,500 vehicles per day, yet there are nearly 140,000 miles of pavement built for such a volume of traffic. It is doubtful if even in the next ten years there will be developed enough traffic on the extra 90,000 miles to justify this over-investment.

If the expenditure of funds had been made with due regard to traffic, there could have been some 300,000 miles of improved intermediate-type surfaces (all adequate to present needs) in place of this 90,000 miles. However, this is all in the past, and a part of the over-design may be properly attributed to temporary inability to gage traffic growth. For the future, however, there is every reason why more exact matching of traffic need to surface type should be obtained.

Summary

It is of course impossible to take care of every situation by simple reference to a chart, but the general principles of a road paying for itself as set forth therein is a useful guide in planning the development of a highway system. The volume of traffic using any particular road is now quite definitely known, and the likelihood of any unusual increase in volume over the next few years is extremely remote, first, because there is a tendency for stabilization in respect to the total number of vehicles using all roads, and second, because with the improvement of an increasing mileage of roads there tends to be a dispersion of traffic, in many cases even a decline in traffic on principal routes which a few years ago were obliged to carry all the travel.

It is unquestionably true that by far the greater mileage of all roads now justifying improved surfaces will be included in the group under 1,500 vehicles per day, and for this volume of traffic the low-cost types have already proved themselves to be entirely adequate. There is no reason why, with the funds now available (provided they are not diverted to other than highway uses), approximately one-third of all road mileage may be improved to a dustless, mudless condition the year around. This third carries more than 90 per cent of all traffic, so that once it has been adequately provided for the highway problem will be in a fair way to solution.

No Method Yet Developed to Fireproof Wood Fully

EXPERIMENTS in fireproofing wood by impregnation with various chemicals have been carried on at the U. S. Forest Products Laboratory for several years. Annually these tests are reported at the meeting of the American Wood Preservers Association. None of the tests so far made has been fully satisfactory.

At the last convention in Chicago a report was made on preliminary tests on six additional single chemicals and 34 combinations of chemicals and more intensive tests on seven combinations. The report was made by T. R. Truax, C. A. Harrison and R. H. Baechler, of the laboratory. Previous work has dealt mainly with single chemicals.

In preliminary or extensive surveys the test specimens are cut from southern yellow pine boards before treatment, impregnated with the chemical solutions under pressure, dried and tested without resurfacing. The specimens are treated with two concentrations of solutions of each chemical, intended to give absorptions of approximately 1 and 5 lb., respectively, of anhydrous chemical per cubic foot of wood. In the intensive surveys the wood is treated in board form with five concentrations of each chemical, intended to give absorptions of 1, 2, 3, 5 and 7 to 8 lb., respectively, of anhydrous chemical per cubic foot of wood. The boards are redried, surfaced lightly to uniform thickness and cut into test specimens. In both series the test specimens are conditioned to approximately constant weight under controlled atmospheric conditions that bring untreated wood to a moisture content of 6 to 7 per cent. Moisture-content determinations are made on samples cut immediately before the fire tests.

The fire-tube test has been used as the measure of fire resistance in both series. In this test loss in weight of the specimen and temperature at the top of the fire tube are the principal measures of fire resistance. Since readings of both conditions are taken at regular intervals, the rate of burning, as well as the total loss in weight and maximum temperature, can be computed. Observations are also made on the tendency of the specimens to glow after the flame dies out.

Of the 67 individual chemicals tested up to this time only four have shown a high degree of effectiveness in reducing both flaming and glowing. They are diammonium phosphate, monammonium phosphate, phosphoric acid and monobasic magnesium phosphate. Several others, such as aluminum sulphate, ammonium bromide and ammonium chloride, were effective in stopping combustion of both types when present in heavy absorptions, but not when in light absorptions. Something like half of the single chemicals tested have shown a distinct effect in reducing the tendency of wood to burn by flam-

ing, and a somewhat smaller number the tendency of wood to burn by glowing.

Analysis of the data on the various chemicals tested indicates that no single compound is an ideal fire retardant. None of the single chemicals tested appears to possess in combination a high degree of fire resistance, low cost, resistance to leaching and freedom from objectionable features such as corrosion of metals, interference with gluing and finishing and injury to the strength and other desirable properties of the wood. The ammonium phosphates in concentrations great enough to give a high degree of fire resistance, for example, add too much to the cost of wood for most uses, interfere to some extent with gluing, are somewhat corrosive to metals and leach easily. Apparently, cheap, effective, permanent treatments that are not injurious to the wood nor objectionable in use can most easily be found in combination of chemicals. The ammonium phosphates are among the most effective single chemicals thus far tested. Ammonium arsenate and ammonium borate, while unusually effective, are somewhat less so than might be expected. Some work has also been done on zinc borate, showing that it has considerable fire resistance.

Of the 34 combinations of chemicals studied, some of the treatments were made in a single impregnation by combining the chemicals in the treating solution. In others an absorption of solution was followed by an ammonia-gas treatment. In still others the wood was impregnated with one solution, dried and then re-impregnated with a second solution.

The first effort to utilize combinations of the more effective acid and basic radicals that are usually insoluble in water consisted in forming low-solubility compounds in the wood by treatments involving two solutions.

The second general method of attack, using compounds of the more effective radicals, involved the primary or monobasic form of phosphate, arsenate, etc., of various basic elements. Several treatments were made involving this principle. Since the monobasic form of these compounds is either not readily available in technical grade or is expensive, most of the treatments were made with other chemicals that produced the soluble monobasic form. Several other solutions containing magnesium and phosphate ions were used.

A number of the treatments, involving a combination of chemicals in one solution, were followed by a treatment with ammonia gas. The purposes of the ammonia-gas treatment were to produce in the wood certain chemical compounds of low solubility that would be resistant to leaching, to reduce the acidity of the treated wood, and to increase the total fire-retardant effect by the chemical fixation of ammonia.

Driving Long Z-Sheeting for Deep-Water Bulkhead

Pier fill in 37½ ft. of water held by single row of Z-section sheetpiles back-anchored to deadmen in fill—Anchor rods installed by horizontal driving through fill

By F. R. Harris
Consulting Engineer, New York, N. Y.

THE HIGH flexual strength of a sheeting of Z-section steel piles was utilized, by developing a successful method of driving, to hold a pier fill against deep excavation alongside in the recent relocation of a floating drydock at the Tebo plant of the Todd Drydock Engineering & Repair Corp. in Brooklyn, N. Y. A plan and sections of the work are shown by Fig. 1. Briefly, the work was to dredge the slip between piers A and B to secure a 37½-ft. depth of water, at about 15 ft. from the face of pier A, to accommodate the drydock. To permit this depth of water so close to the face of pier A, it was necessary to rebuild the pier.

The method selected was to drive steel sheetpiling close to the face of the existing pier and to anchor it back to vertical and batter timber piles and reinforced-concrete blocks. The length of pier A reconstructed was 618 ft., and 175 ft. was the distance across the head of the slip between piers A and B. For the greater portion of the bulkhead along the face of pier A only one set of anchors was used; but for the portion immediately in front of the storehouse two sets were used, because it was found that the first set could not be driven to the required depth to insure safety.

Selection of Z-piles

The development of steel sheetpiles has been gradual and apparently started with the rolling of the straight web section, which is of great service when used in cellular cofferdams or where the stress on the piling is principally tension parallel to the web. The only need for additional metal in this section other than that required for the tension is that required to permit it to be handled and driven. The straight web section is of little use where strength in flexure perpendicular to the face of the sheetpiling is required.

To supply this strength, the natural step was to alter the web so that this was deepened and the sheet assumed a trough shape. As the needs for greater flexural strength grew, the troughs were made still deeper. In all cases, however, the troughs were alternately placed so as to increase the distance between their backs, and in this way increase the flexural strength of the two units when acting together. This natural development has left the interlock at the neutral axis of the two sheets.

Attempts have been made from time to time to overcome these objections by the rolling of a Z-shaped pile, which would at once place the metal of the interlock in the most advantageous position to give the section flexural strength in the direction of the wall, and at the same time effectively connect the flanges with the web of the unit.

With apparent advantages, it is strange that no more extended use has been made of the Z-shaped piles. One of the reasons usually advanced is that this pile is difficult or impossible to drive in compact ground, as the piles are apt to twist.

Fig. 1—Outline plan and sections of pier, showing Z-pile bulkhead and method of back-anchoring it to the retained earthfill.

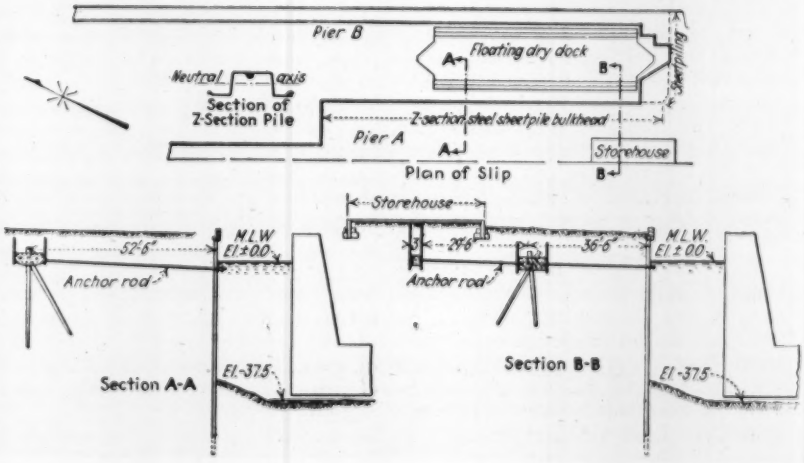


Fig. 2—Method of driving Z-piles in pairs in successive stages.

In the building of this bulkhead, economy in design was an essential consideration. In addition to economy, the Z-section supplied the required strength without resorting to welding the interlock or adding strength by welding or riveting steel members to the pile.

Driving method developed

In view of the lack of previous experience in driving, special attention was paid to this phase of the work. The crew selected had had considerable experience in driving steel sheetpiles of the trough type, but the Z-section was new to them. It was not difficult, however, to get them adapted to the novel section.

The sheets were interlocked, two at a time, on the ground so as to form a trough and, to hold them in this position, ¼x1½-in. metal straps were spot-welded to the two outstanding flanges of the Z, thus avoiding the use of special clamps commonly employed. The straps were lighter, easier to install and could be left in place, whereas the clamps were heavy and difficult to set and remove.

At first, three straps were welded to each two sheets forming the trough, one near the bottom, one close to the center, and the third near the top. Later, as driving progressed, it was found best to do away with the two lower straps and retain only the upper strap.

After threading the two Z-piles together, these were set up in the manner usually followed in setting up trough-shaped sheetpiles, tapping them down until their driving became hard. In this way, driving through hard material was avoided before the sheets had been set in place and interlocked one with the other.

The sheets driven varied from 45 to 60 ft. in length and penetrated first through a layer of clay and silt about 10 ft. thick, and then through sand varying from fine to coarse and in places becoming coarse gravel. The depth driven through the sand and gravel varied from 25 to 40 ft. In spite of this, no real difficulty was experienced in driving the Z-shaped piling.

As stated above, two sheets forming a trough section were driven at the same time, using at first a No. 7 hammer. The sheets were driven in succession for a depth of about 5 to 6 ft., and no attempt was made to force any pair more than about 6 ft. below the adjacent higher sheets. In this manner little opportunity was given to the sheets to twist, if such a tendency existed. When the two sheets had gone down this distance beyond the adjacent higher sheets, the driving became hard, taking about 35 to 40 blows to the inch. Pursuing this method, the sheets were driven for a depth of 15 to 20 ft., the No. 7 hammer being used. They were driven the remaining distance of 20 to 25 ft. with a No. 9B hammer, following the same principle described above.

In some spots the driving became hard, taking 50 to 60 blows to 1 in. In such cases a water jet was employed, but the point of the jet about 10 ft. above the bottom of the sheetpiles was maintained. This facilitated the driving considerably, especially where gravel was encountered.

It was necessary to make a special steel head to fit the hammer and also the double piles. It was also found necessary to fit guides to the hammer that would permit it to rest on the sheets without the necessity of holding it in place with the hoisting line. By this means it was possible to deliver the full and effective blow of the hammer to the piles.

The last few feet of driving were naturally the hardest. There were also a few pairs that could not be driven with the heavier hammer. To get these sheets down to the required elevation, a special cast-steel head was made and was fitted to only one of the piles instead of to the pair. This head also bore directly on the ram of the hammer and on the piling. With this equipment it was possible to drive the piles down to the required depth.

Little brooming of the tops of the piles was experienced, and the maximum length found necessary to be cut off because of brooming was about 1 ft. No difficulty was experienced in maintaining the sheets in line, both in the vertical or horizontal direction, and no V-shaped piles had to be driven to maintain the lines. In some cases in setting the sheets up, it was found necessary to cut the bottom on a bevel to maintain the line.

After the bulkhead had been anchored and the dredging completed, a diver was sent down to examine each interlock joint. Only one interlock joint was reported open, and this had an opening about ¾ in. wide at its maximum point and tapering to zero. The length of the opening was about 3 ft. Considering the exceptionally hard driving, and in view of the experience on other types of sheetpiling, this was considered very satisfactory.

Fig. 3—Top of Z-pile wall driven to grade and ready to be capped with pier string piece.

Placing anchor rods

An unusual feature in this work was the method used in placing the anchor rods. The existing pier A consisted of a timber sheetpile bulkhead with timber anchor piles and steel anchor ties, back-filled to grade, with a depth of water in front varying from 5 to 15 ft. Instead of excavating trenches to place the new anchor rods, the trench for the timber-pile anchors was excavated parallel to the proposed bulkhead, and the anchor rods were driven horizontally from the water side until they projected into the trench. Afterwards they were adjusted to the steel channel wales by jacking, heavy plate washers were screwed, and this was all embedded in the reinforced-concrete anchorage.

To accomplish this, the steel rods were protected with a driving point and driving head, and by means of a movable platform attached to the floating pile driving rig they were driven into the bank with a No. 7 hammer. The driving was facilitated by having the 54-ft.-long rods provided with turnbuckles at their centers. In this way the unsupported length of the rod to be driven was much shorter.

The scheme worked admirably, with consequent saving in excavating and backfilling. Only twice was an obstruction encountered, and in these cases the ground was excavated immediately above the point of the rod and the obstruction removed. The maximum drift of the point of the rods from the position intended was 1 ft. Little difficulty was experienced in bringing the rods to the desired position, as the material through which they were driven permitted the tierod to be brought into position with little difficulty.

The work was designed by the writer, who also acted as consulting engineer to Construction Management, Inc., the contractor.

Wood-Beam Design Method Promises Economies

Tests and analyses show horizontal shear stresses to be lower than usually assumed, by virtue of top and bottom halves of a beam acting as separate units—New design method takes this two-beam action into account

By J. A. Newlin, G. E. Heck, and H. W. March

Forest Products Laboratory, U. S. Department of Agriculture, Madison, Wis.

A NEW design method for calculating the horizontal shear in wooden beams, developed by the Forest Products Laboratory through mathematical analysis and tests, assumes that, because of the shear distortion in the vicinity of the base of checks or fissures that are present in all beams, the upper and lower halves of the beam act to some extent as independent beams. The result is to relieve the mean shearing stress in the neutral plane, and since this reduced shearing stress is not considered in present design methods, they are often over-conservative and therefore uneconomical to use. In certain cases, such as floor beams of highway bridges and railway ties, usual design methods predict stresses that are two or three times the ultimate shearing stress of the material. Still these members are carrying their loads without failure. In the discussion presented here an attempt is made to explain the elastic behavior of a checked beam under load and thus to explain the discrepancy existing between the facts of experience and the predictions of the usual methods of calculating shear. The background for this explanation is furnished by an approximate mathematical analysis of the problem combined with the results of a series of about 200 tests, in which the loads causing shearing failure were observed on built-up artificially checked beams varying from ¾ x 1½ in. to 8 x 16 in. in cross-section and with varying amounts of checking. On the basis of the theory and the results of tests, practical directions are given for the more realistic calculation of loads that will cause shearing failure.

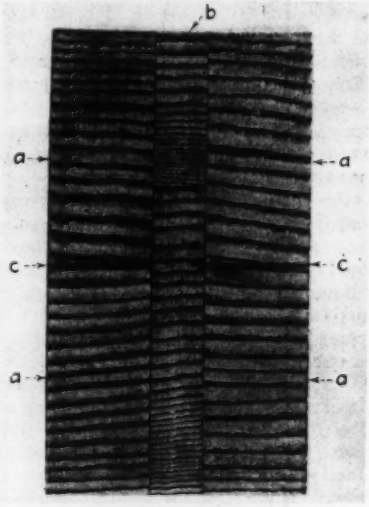
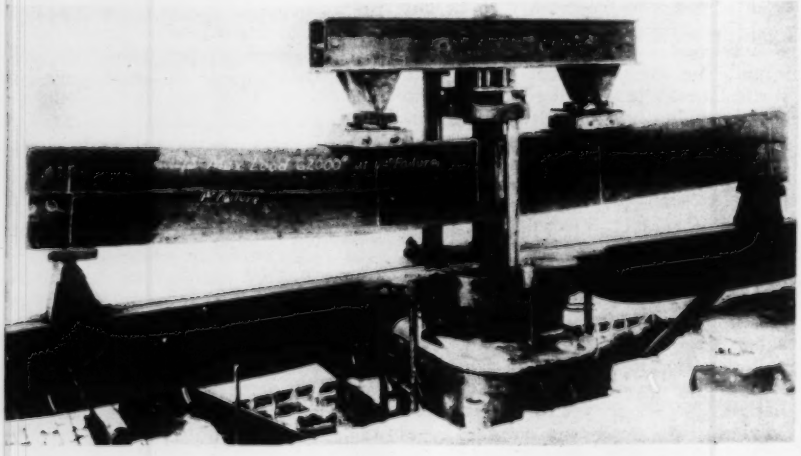


Fig. 1—Cross-section of a built-up artificially checked beam such as was used in the test. The four parts *a* were glued to the central portion or web *b*. The joints *c* were paraffined to prevent sticking of any glue that might protrude from adjacent joints and to minimize friction.

Fig. 2—Characteristic failure of a timber beam, failure occurring in horizontal shear in the neighborhood of the neutral axis.



In the tests, built-up artificially checked beams of carefully matched material were used. A typical section is shown in Fig. 1. The checks are placed in the middle of the lateral faces, as this is the position in which they are most likely to occur and to cause failure by shear. A characteristic failure, by horizontal shear, of a naturally checked structural timber loaded at two points is shown in Fig 2. In all other tests referred to in this paper the beams were loaded at one point only.

Two-beam action

An explanation of the behavior of a checked beam is found in the fact that, owing to the shear distortion in the material in the vicinity of the base of the checks, the upper and lower halves of the beam act to some extent as independent beams. An approximate mathematical analysis indicates a vertical distribution of shearing stress as shown in Fig. 3 The shearing stress plotted is the mean shearing stress at failure averaged over the full width of the beam. The existence of the "two-beam action" is evident from the curves. The mean shearing stress in the neutral plane is relieved by the action of the upper and lower halves of the beam as partly independent beams. The significance of this action is emphasized by the further result of the mathematical analysis that the reaction R at the support nearer the load may be expressed as the sum of two terms, of which only the first is associated with shearing stress in the neutral plane. Thus it is found that

$$R = B + A/a^2 \qquad (1)$$

where

$$B = \tfrac{3}{4} Jbh, \qquad (2)$$

J being the mean of the shearing stress in the neutral plane over the full width of the beam, and b and h being the width and depth, respectively, of the beam. The portion B of the reaction is precisely the reaction that would be associated with the mean shearing stress J in the neutral plane in the usual theory of beams and may be referred to as the "single-beam portion" of the reaction.

In the second term of equation 1, a is the distance from the load to the nearer support, and A is a quantity determined chiefly by the dimensions of the beam, the longitudinal Young's modulus, and the mean longitudinal displacement on the lower face of the upper half of the beam at points immediately over the support. The portion of the reaction expressed by this term is attributed to the independent action of the upper and lower halves of the beam and is not associated with shear in the neutral plane. It is the "two-beam portion" of the reaction. The values of the two-beam and single-beam portions of the reaction at the

instant of failure were determined by testing a large number of beams and by combining the results of tests of pairs of carefully matched beams loaded to failure, the concentrated loads being applied at different points. On entering the results of each pair of tests in equation 1 two equations were obtained that could be solved for B and A.

Test results suggest design procedure

In this procedure it was assumed that the values of A and of the mean shearing stress J in the neutral plane at failure were independent of the position of the load. The justification of both assumptions is found in the fact that the results of the tests were represented approximately by equation 1 with A and B as constants for the whole series. The tests showed that as a concentrated load approaches a support, the two-beam portion of the reaction increases rapidly, while the single-beam portion associated with shear in the neutral plane, remains practically constant and becomes a correspondingly smaller fraction of the reaction. As a consequence, the point of application of the minimum concentrated load to produce failure by shear is not just inside the support, as is commonly assumed because of the usual simple beam theory, but is at some distance from the support. The results of the tests of three series of beams are given in the accompanying table and are represented graphically in Fig. 4. The single-beam and two-beam portions of the reaction shown in the table were computed by combining in pairs the mean reaction corresponding to $a = 7$ with that corresponding to each of the other points of loading. If these reactions are combined in pairs in other ways, results varying somewhat from those shown may be obtained.

The results of the numerous tests may be summarized in the following statements: For checked beams with a span-depth ratio of 9 to 1 the point of application of the minimum concentrated load causing failure by shear is at a distance from the support approximately three times the height of the beam. The distance of the critical point from the support is somewhat greater for longer spans and somewhat less for shorter spans; but in any case, for loads applied at three times the height of the beam from the support, the two-beam portion of the reaction at the nearer support is approximately one-sixth.

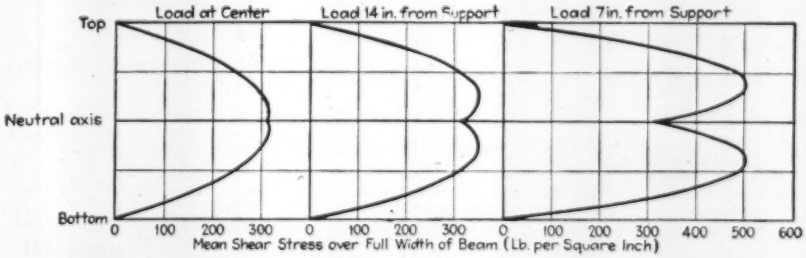

Load at Center
Load 14 in. from Support
Load 7 in. from Support
Top
Neutral axis
Bottom
0 100 200 300 0 100 200 300 0 100 200 300 400 500 600
Mean Shear Stress over Full Width of Beam (Lb. per Square Inch)

Fig. 3—Theoretical variation of horizontal shear stress in checked beams for various positions of a single concentrated load. Beams were 2½x4½x45 in. on a span of 42 in. The depth of the checks was 1 in.

Design recommendations

1. If there are moving loads, place them so that the heaviest concentrated moving load is at a distance from the support of three times the height of the beam. After this has been done the following recommendations apply both to static loads and to the moving loads.

2. Neglect all concentrated loads within a distance from the support equal to the height of the beam.

3. Consider all concentrated loads that are from one to three times the height of the beam from the support as being at three times the height of the beam from the support and compute the resulting reaction. Neglect one-sixth of the reaction. This one-sixth is the two-beam portion of the reaction and is not associated with shear in the neutral plane.

[Note: For very small span-depth ratios (less than 6 to 1), place the loads designated in recommendations 1 and 3 above, at the *middle* of the span.]

4. Consider all other concentrated loads in the usual manner.

5. Neglect 10 per cent. of the uniform load. This 10 per cent is taken care of by two-beam action.

6. Assume lateral distribution of the various loads to adjacent parallel beams or stringers on the basis of their assumed placement.

7. Calculate the shearing stress in the neutral plane by the usual formula, using the full width of the beam and all of the reaction, obtained as directed, that is not included in the two-beam portion.

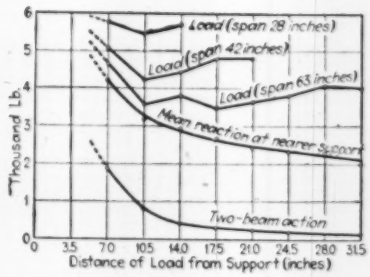

Load (span 28 inches)
Load (span 42 inches)
Load (span 63 inches)
Mean reaction at nearer support
Two-beam action
Thousand Lb.
0 1 2 3 4 5 6
0 35 70 105 140 175 210 245 280 315
Distance of Load from Support (inches)

Fig. 4—Test results of built-up artificially checked beams of various lengths. Note the relative values of the single-beam and two-beam portions of the reaction and the increasing importance of the latter as the load is placed nearer the support.

8. Use 90 per cent of the safe shearing stresses previously recommended by the Forest Products Laboratory, since these recommendations did not take into account the effect of two-beam action.

The use of these recommendations will result in a considerable saving of material. The highway bridge, on account of the effects of lateral distribution of the loads to adjacent stringers, is probably the structure that will show the greatest difference in the results of the application of the new and old methods. An application of the two methods of calculation to a highway bridge follows:

Consider a single concentrated load P moving along a floor beam or joist. Assume the span to be 16 ft., the height of the joist, h, 16 in., and its width, b, 5 in., and all dimensions full nominal size. Assume also that when the load is in the center of the span the floor carries one-fourth of it to each of the two adjacent beams, one on either side. In such a case it has been calculated that the side beams will carry a little more than 40 per cent of the load when the load is at three times the height of the beam from the support. The shearing stress, J, in the neutral plane is given by

$$J = \frac{3}{2} \times \frac{5}{6} \times \frac{R}{bh} \qquad (4)$$

where R = reaction due to the load P placed at three times the height of the beam from the support.

But, according to the assumptions as to lateral distribution,

$$R = \frac{6}{10} \times \frac{3}{4} \times P = \frac{9}{20} P \qquad (5)$$

Assuming 100 lb. per sq. in. as the tabular value for the safe shearing

RESULTS OF TESTS OF BUILT-UP, ARTIFICIALLY CHECKED SPRUCE BEAMS
(Various lengths; breadth, 2¼ in.; height 4½ in.; depth of checks, 1 in)

Distance (a) From Load to Nearer Support In.	Load at Failure: 28-In. Span Lb.	42-In. Span Lb.	63-In. Span Lb.	Mean Reaction at Nearer Support Lb.	Single-Beam Action Lb.	Two-Beam Action Lb	Two-Beam Action × a^2
7.0	5,770	5,050	4,660	4,226	....	1,872	91,700
10.5	5,460	4,240	3,560	3,186	2,354	832	91,700
14.0	5,680	4,430	3,780	2,911	2,473	438	85,800
17.5	....	4,800	3,460	2,649	2,349	300	91,900
21.0	....	4,800	3,630	2,410	2,183	227	100,100
24.5	....	....	3,780	2,310	2,140	170	102,000
28.0	....	....	4,050	..... Failed in tension			
31.5	....	....	3,970	1,985	1,869	116	115,100

stress and reducing it by 10 per cent as explained earlier, it follows from (4) and (5) that

$$90 = \frac{3}{2} \times \frac{5}{6} \times \frac{9}{20} \times \frac{P}{5 \times 16} \quad (6)$$

Hence, $P = 12{,}800$ lb., the safe load in shear.

Under the old assumption that maximum shear in the neutral plane will occur as the heavy wheel leaves the span, using the full tabular value of 100 lb. per sq. in. as the safe shearing stress, and noting that at the end of the beam there is no lateral distribution, the safe load P is found from

$$100 = \frac{3}{2} \times \frac{P}{5 \times 16}$$

From which $P = 5{,}333$ lb.

This method of computation leads to a safe load less than one-half that permitted by the new method.

In the case of a single beam with no lateral distribution, we omit the factor 6/10 from equations 5 and 6, obtaining

$$R = \frac{3}{4} P$$

and

$$90 = \frac{3}{2} \times \frac{5}{6} \times \frac{3}{4} \times \frac{P}{5 \times 16}$$

Then $P = 7{,}680$ lb.

This load is more than 40 per cent greater than that permitted by the old method of calculation.

Bridge Welding Practices in Australia

Several structures of respectable size built in Victoria—Procedure characterized by use of covered electrodes and multiple-run beads

By W. T. B. McCormack
Chairman,
Victorian County Roads Board, Australia

SINCE 1928 three arc-welded bridges have been built in Victoria, Australia, and welding was used for centering trusses on a high concrete structure. These bridges compare favorably in size with welded bridges built elsewhere in the world. Alternate riveted and welded designs were prepared, and in each case the lowest bid was for a welded structure. From the building of these bridges and other types of structures a successful procedure has evolved which is placing welding on a par with riveting.

In Australia practically all structural welding has employed covered electrodes. It has also been found that the desirable characteristics of welds with regard to penetration, ductility, uniformity and strength require the multiple-run system of welding. Covered electrodes of 8 and 10 gage, in lengths of 18 in., form the basis of design, with welding currents of 100 to 150 amp., depending upon the thickness of the metals to be joined. The initial run is usually of a 10-gage electrode deposited in a length of 7 to 9 in., forming a ⅜-in. triangular fillet. Where greater strength is required, additional runs are made. The standards for maximum fillets are shown in the accompanying table. Definite strengths being obtainable from the combinations of multiple runs, no attempt is made to specify fillet size. Although there have been attempts lately to introduce fillet size as a standard of measurement, it has not been found essential nor desirable. If a weld fillet is specified by size, the psychological effect upon the welder is to build up quickly with heavy runs, whereas greater benefits at no extra cost are obtained by a larger number of runs.

Welding costs about one shilling per foot of ⅜-in. fillet. The ratio of cost to strength provided increases very rapidly with large fillets and is three times as high for a ¾-in. fillet as for a ⅜-in. fillet. Therefore, it is more economical to use a greater length of small fillet. Furthermore, large fillets generally are not economical, as it is considered essential to weld plates all around the joints to prevent rusting between them. The loads in members of highway

Fig. 2—Four-span continuous welded truss bridge over Sunday Creek. Chords of trusses are two channels spread apart to permit insertion of 6-in.-wide I-sections as vertical members. Diagonals are flat bars.

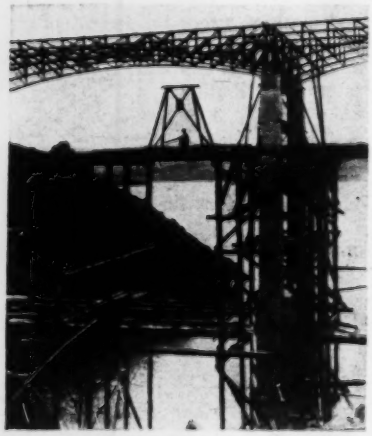


Fig. 1—Welded trusses in Pykes Creek Bridge used to support forms for concrete ribs and to supplement the rod reinforcing.

STANDARDS FOR WELDING DESIGN, VICTORIA, AUSTRALIA

Plate Thickness (In.)	No. of Runs	Size of Electrode (S.W.G.)	Current (Amp.)	Length of Run Per Electrode (In.)	Ft. of Electrode Per Ft. of Welding	Ult. Strength (Tons Per Lin. Inch of Welding) End Welds	Ult. Strength (Tons Per Lin. Inch of Welding) Side Welds
⅛	1	14	55	15	1.2	1.0	0.6
¼	1	12	75	13	1.4	2.0	1.9
	or 1	10	100	18	1.0		
⅜	1	10	100	13	1.4	3.0	2.2
½	1	10	100	9	2.0	4.0	2.8
⅝	2	10	100	9	4.0	4.8	3.4
¾	3	10	100	9	6.0	5.6	4.0
¾	1 + 2	10	100	9	2.0	7.2	5.2
		8	125	8	5.0		
⅞	1 + 4	10	105	9	2.0	9.0	6.4
		8	130	9	8.0		
1	1 + 5	10	105	9	2.0	10.4	7.2
		8	130	8	11.0		

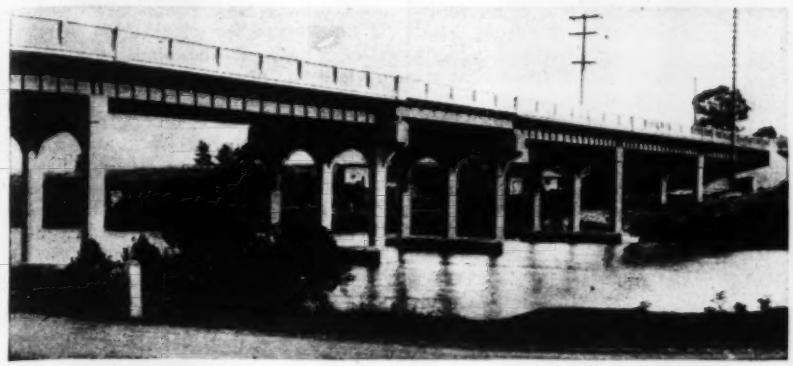


Fig. 3—Welded plate-girder bridge over Swan Reach. Short center span of rolled beams is planned for a future lift span. Three spans either side of this center span are designed as cantilevers with a suspended span in the center.

bridges being relatively small, large fillets are seldom necessary. Where reasonable fillet sizes cannot provide the necessary strength, the end of the member may be sawtoothed to increase the length of weld.

Pykes Creek Bridge

Of the four bridges erected to date, the first was designed in 1928 and constructed in 1929. It is not a welded bridge in the strict sense of the word but a reinforced-concrete structure in which light welded trusses were used to support the forms, concrete and the bar reinforcing. These trusses are placed within the beam stems and comprise a part of the steel reinforcement. Their weight is 22.65 tons.

This bridge, known as the Pykes Creek Bridge, is 246 ft. long and 28 ft. wide between parapets. The length is made up of two 40-ft. end spans, two 54½-ft. intermediate spans and a 56½-ft. center span. The bridge crosses a deep ravine that is part of a water-storage reservoir and is 90 ft. high to the roadway. Foundations are on rock, and the superstructure is continuous with variable moments of inertia. Fig. 1 shows the completed trusses in position, ready for the pouring of the concrete.

Sunday Creek Bridge

The Sunday Creek Bridge, designed and constructed in 1930, is also founded on rock and is a continuous steel truss carrying a concrete deck 22 ft. wide (Fig. 2). Spans are 52 ft. at the ends and 65 ft. for the two central spans, which arrangement gives approximately equal positive moments in all spans and equal negative moments at the piers. Abutment supports are on rollers. Three lines of trusses are used. Chords consist of pairs of angles separated 6 in., permitting 8x6, 6x5 and 6x3-in. I-sections to be used as verticals. Except where stress reversals occur, the inclined web members are pairs of flats welded to the outside of the chord angles. The trusses were shop-fabricated in lengths of 30 to 35 ft. and field-spliced at about the quarter points. These splices were made by means of turned bolts driven into reamed holes, although it is now recognized that a field-welding plant would have made splices more economically. The total weight of structural steel in the bridge was 48¾ tons.

Swan Reach Bridge

Designed and constructed in 1931, the Swan Reach Bridge is a structure with a total length of 406 ft. made up of six welded plate-girder spans of 62 ft. and a central span of 43 ft. Rolled beams were used in the central span, and it is designed as a future lift span (Fig. 3). Foundations are piles driven into silt. Each section of three spans, in the length of 186 ft. each side of the lift span, consists of cantilevered plate girders and a suspended span of 34 ft.

In this bridge, as in the Pykes Creek Bridge, the slab is constructed as a vehicular deck 27 ft. wide, including a 5-ft.-wide gravel sidewalk held between an outer curb and a concrete curb cast on the deck. This arrangement gives flexibility of roadway width at little extra cost.

The details of the welded plate-girder section and the cantilever-suspended-span junction are shown in Fig. 4. Flange angles are eliminated. Stiffeners are flats welded intermittently on both sides to the web plate and solidly at the ends to the flange plates. At the suspended-span junction the leg of the angle paralled with the girder web was hand-dressed to a radius of 36 in. and supported on a corresponding leg dressed flat, with a strip of brass ⅛ in. thick separating the two angles.

Fig. 4—Junction of cantilever and suspended spans on Swan Reach plate-girder bridge.

The total weight of welded steel in the bridge was 101 tons. A total of 16,000 ft. of welding was used, consisting of one run of 10-gage electrode 18 in. long deposited in 9 in. and known as 10/9.

Snowy River Bridge

The Snowy River Bridge, designed in 1931 and constructed in 1932, is situated on a mountain road 95 miles from the nearest seaport and is without railway facilities. It was built essentially for livestock (cattle and sheep) but will safely carry a 10-ton vehicle. The live load was assumed at 100 lb. per sq.ft., plus 30 per cent impact. The total

Fig. 5—Snowy River Bridge, good for 10-ton vehicles, comprises two spans of 105 ft. and four of 135 ft. Trusses are 10 ft. deep, 10 ft. apart and designed as continuous with a fixed bearing on center pier.

length of the bridge is 750 ft., and the average height from deck level to streambed level is 56 ft.

There are two end spans of 105 ft. and four intermediate spans of 135 ft. The trusses were made continuous throughout, with a fixed connection on the center pier and rollers on the other piers and the abutments. Truss fabrication and erection was necessarily done at the site, each span being assembled under its final position.

The welding plant consisted of a gasoline-driven three-phase generator and three welding sets. The maximum length of members possible to transport over the road (12-per cent grades and 20-ft.-radius curves) was 40 ft.

Trusses were built up almost entirely of channels except for minor bracing, for which angles were used. The trusses are Warren, deck-type, 10 ft. deep and set 10 ft. apart on centers. Panels are 15 ft. long. Chords, consisting of pairs of channels 12⅛ in. clear between webs with flanges turned out, are 10 in.x24.46 lb. at the piers and 9 in.x17.46 lb. toward the center of the lower chord. Web members are pairs of channels with flanges turned in, fitted between the chord channels and welded directly to them. Compression sections were laced with 2x⅜-in. and 2½x⅜-in. straps, which were welded to the channels.

The trusses were erected as simple spans. To put in the proper dead-load stress for continuous-truss conditions, the splices between the trusses on the piers were made by placing a hinge in the bottom chord and by pulling the top chords together with a tension of about 40,000 lb. per truss. The tension was applied with screwed 1¼-in.-diameter rods, and nuts were placed between temporary pieces connecting the top-chord channels. The rods were calibrated for elasticity, and the tension was determined by extensometer. The total weight of steel was 132.7 tons. A total of 20,328 lin.ft. of welding was applied, consisting of one run of 10-gage electrode deposited in 9 in. and one run of 8-gage electrode deposited in 8 in. The minimum weld run was a single 10/9, and the maximum was 10/9 plus 2 at 8/8. The bridge during construction is shown in Fig. 5.

Organization

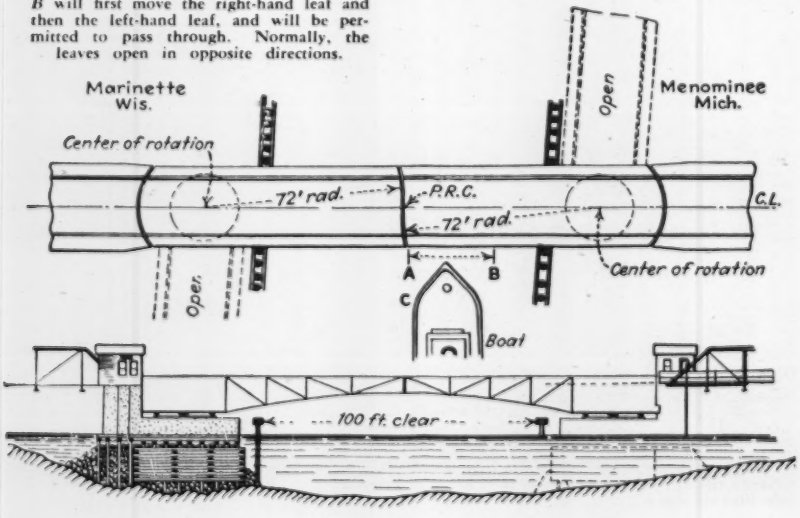
The bridges were built by the Victorian County road board by contract from funds raised on vehicle and gasoline taxes. Construction was under the general direction of Louis F. Loder, chief engineer for the board. The designs were prepared and the construction work was suspervised under the direction of Marsden G. Dempster, bridge engineer.

Letters to the Editor

A Swing-Bascule Bridge

Sir—The article on the unusual foreign bridges (*ENR*, April 20, 1933, p. 498) affords opportunity to call attention to a bascule bridge of rather unorthodox design in which opening is effected by swinging the leaves in a horizontal instead of a vertical plane. It was built in 1926 after the cities of Menominee, Mich., and Marinette, Wis., had been ordered by the War Department to provide a clear opening for navigation of 100 ft. in lieu of that provided by the then-existing center-bearing swing span over the Menominee River. As the estimated cost of a standard bascule bridge was far in excess of the amount for which this city could issue bonds at the time, the writer devised a plan which he estimated could be constructed within our means. Owing to the departure from general design, we engaged Klug and Smith, consulting engineers of Milwaukee, who worked out the structural details, and the bridge was built by the Wisconsin Bridge & Iron Co. and the Greiling Engineering Co., of Green Bay, as contractors. The bridge has now been in operation about four years, or long enough to demonstrate its practicability.

Sketch of twin-leaf swing bridge over the Menominee River in Michigan. A boat colliding with the bridge between *A* and *B* will first move the right-hand leaf and then the left-hand leaf, and will be permitted to pass through. Normally, the leaves open in opposite directions.



It will be noted that the ends of the leaves form a reversed curve that permits the leaves to open without fouling one another. In normal operation the leaves are swung in opposite directions. However, both leaves may swing in the same direction provided one starts slightly ahead of the other; and one of the particular features is that in case a boat collides with it, the bridge can open automatically, if the locks have been released. Thus, as shown in the illustration, if a boat collides with the bridge between *A* and *B* (and boat captains are advised to steer for this area) the right-hand leaf will be pushed ahead so that when *C*, the bow of the boat, reaches *A*, the other leaf can be shoved in the same direction, permitting the boat to pass through.

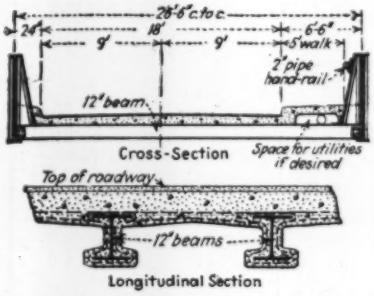
The center lock consists of bars 4x6 in. driven into sockets in the ends of the leaves. At present this is a manual operation. The tails of the leaves are held rigid by wedges, and in the center of each tail there is a lock bar which is caused to fall into place by the action of a cam timed to act with the wedges. Provision is made to operate the locks by motors, although these have not yet been installed. The swinging of the bridge is effected by 15-hp. motors.

At the time the bridge was being considered we examined numerous records to find if it infringed on any patents. One such patent was found, issued some 30 to 35 years ago, and we also found that a bridge of similar type had been built in Sorento, Italy (*Engineering News*, Sept. 13, 1890, p. 227). In this Italian bridge, however, the leaves pulled back before swinging.

Menominee, Mich., March 21, 1933. T. R. HALSEY.

Shallow-Deck Bridges

Sir—In the April 6 issue of *Engineering News-Record*, p. 430, there appears an article descriptive of a highway bridge built under limiting conditions. As similar conditions are often met with in both highway and railway bridge work, the writer wishes to call attention to a type of design that provides a more shallow deck than the one shown in the article referred to. While weight and spacing of floor beams are not indicated in the accompanying drawing (these, of course, depending



Suggested design for a bridge of shallow depth, giving maximum headroom below.

upon the loading and length of beams that the bridge is designed for), it will be found that for ordinary highway bridges having a width between main girders, as shown, and for 20-ton truck loading, two lanes of traffic and a 5-ft. sidewalk, 12-in wide flanged floor beams may be used. It will be noticed that this type of deck has about 8 in. less depth than the type shown in the article referred to. If it is desired to incase the floor beams in concrete, this can be easily done, as shown, either by Guniting or by formwork. Of course, if desired, the main girders can also be incased and treated for a more artistic effect.

Besides providing a more shallow type of deck the through-plate girder bridge also offers a greater factor of safety against automobiles going over the sides.

As to the manner of reinforcing the rolled girders, shown in the April 6 article, it would seem to the writer that welding a reinforcing plate to the bottom flange would have been much better. This method places the reinforcing steel in a more effective position, and the decreased headroom beneath amounts to practically nothing, since the flanges are incased in concrete. The reinforcing plates should be welded in place before the girders are set, thus avoiding the requirement of an up-weld. These plates should be stopped short of the sole plates.

East Cleveland, Ohio, April 19, 1933.

ARTHUR N. DOUD,
Formerly bridge engineer, P. & W. Va. R.R. Co.

Designing for Continuity

Sir—I wish to call to your attention a treatise entitled, "The Analysis of Continuous Frames by the Method of Restraining Stiffnesses," written by Earl B. Russell, structural engineer, San Francisco, Calif., and published by him for the solution of moments in rigid frames and similar structures. Two notable contributions upon this subject have previously been published: "Moments in Restrained and Continuous Beams," by L. H. Nishkian and D. B. Steinman, (*Trans.* Am.Soc.C.E., Vol. 90, 1927) and "Analysis of Continuous Beams by Distributing Fixed-End Moments" by Hardy Cross (*Proceedings*, Am.Soc.C.E., May, 1930). By the first of these methods analysis is done graphically, and by the second it is done by a method of distributing fixed-end moments involving a process of repetition until a balanced condition is attained.

Mr. Russell's method is based upon the restraining stiffnesses of members meeting at a joint; i.e., the moment at an end of a loaded member is a function of the stiffnesses of the other members of the joint relative to the stiffness of the loaded member, that this moment is resisted by the other members of the joint in proportion to their stiffnesses, and that the moments at the far ends of the non-loaded joint members are functions of the stiffnesses of restraining members at those ends. The moments are obtained directly by analytical methods and do not require the tedious solution of simultaneous equations or the repeated process of a series of approximations. The method is based upon the relations set up by slope-deflection equations, and the basic formulas used are derived from them. Three fundamental relations are established, and formulas are given for them for different conditions. The first formula gives the moment relations, the second gives relative stiffness factors, and the third gives the carry-over moments. An additional formula is given to provide correction to the fixed moments for unsymmetrical loading. The formulas need not be used directly, as very complete tables are given for their solutions, and by means of these tables the analysis is simplified.

Part I deals with members in frames having constant moments of inertia. Tables are also included giving fixing moments for trapezoidal, uniform and triangular loading and for partial uniform and triangular loading and concentrated loading. Several examples are given showing the use of the method and tables. Part II treats of members having variable moments of inertia and beams with haunches of different types. The slope-deflection equations are modified to suit members of variable moments of inertia, and the three fundamental formulas are again derived. A very complete set of tables is given covering straight, parabolic and sharply curved haunches. Some of these are based upon the tables given by Walter Ruppel in his discussion of the paper of Messrs. Nishkian and Steinman (*Trans.*, Am. Soc.C.E., Vol. 90). Tables are also given for wide supports, and a discussion of their effects is included. Examples involving members of frames with variable moments of inertia illustrate the use of this method for such cases.

It is believed that this method and accompanying tables will prove of great value to engineers dealing with continuous and rigid frames. Many of the tables will be useful in connection with the use of the method of distributing fixed-end moments.

H. W. BOLIN,
Bolin and Hess, Structural Engineers.

Oakland, Calif., Feb. 8, 1933.

Old Concrete Water Tunnel Inspected After 25 Years

Sir—The description of the inspection of a concrete-lined water tunnel in Chicago (*ENR*, March 23, p. 374) is of unusual interest and value in view of the singularly few records of this sort so far published. The article lacked only one particular item of importance. Would it be possible to publish the average water velocity which the tunnel has experienced during service?

ROBERT F. LEGGET,
Chartered Civil Engineer.

* * *

Sir—Concerning velocities in the Blue Island Ave. tunnel, our records show that the average daily pumpage from this tunnel for many years has been about 70 m.g.d., which would give an average velocity of 2.16 ft. per second. The average daily maximum pumpage for a period of more than ten years has been 88 m.g.d., corresponding to an average maximum velocity of 2.7 ft. per second. The minimum daily pumpage is about 50 m.g.d., which would produce a minimum velocity of about 1.54 ft. per second.

Some hourly pumpage rates and the corresponding velocities are as follows:

Year	Maximum Hourly Pumpage, M.G.D.	Velocity, Ft Per Second
1921	130	3.7
1928	95	2.92
1929	92	2.83
1930	92	2.83
1931	90	2.77
1932	93	2.86

The accompanying table gives data obtained from a survey of this tunnel in November, 1931, by the pumping station efficiency section of the bureau of engineering, Paul Lipper, engineer in charge.

MYRON B. REYNOLDS,
City Engineer.

Chicago, Ill., April 18, 1933.

LOSS OF HEAD AND FRICTION FACTORS
PINE ISLAND AVE. TUNNEL SURVEY, NOV. 18, 1931

Tunnel diameter is 8.0 ft. Rate of flow, 70.11 m.g.d., and equivalent velocity, 2.158 ft. per second.

Section	Length of Section, Ft.	Loss of Head, Ft.	Loss of Head, Ft. Per 1,000 Ft.	Value of *C*, Chezy	Value of *N*, Kutter
Delaware Place, shaft 1 to shaft 3	7,386	1.229	.1663	118.3	.01406
Kingsbury St. shaft 3 to shaft 4	7,048	1.158	.1643	119.0	.01394
Boston Ave. shaft 4 to shaft 11	11,422	1.470	.1287	134.6	.01239
22d St. pump station, shaft 11 to weir well of station	490*	.251	.512	68.2	.....

*(Equiv. lgth. of 8.0 = ft. = diam. tunnel)

During the survey the gate in Jefferson St. was closed, shutting off the 8-ft. connection in Van Buren St. that supplies the Harrison St. pumping station.

Engineering News-Record

EDITOR—F. E. Schmitt. EDITORIAL STAFF—V. T. Boughton, W. G. Bowman, F. W. Herring, C. S. Hill, H. W. Richardson, W. W. De Berard, N. A. Bowers, J. I. Ballard.

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May 11, 1933 Volume 110 No. 19

A Material Gain

THE ADMINISTRATION'S railroad bill as it was submitted to Congress last Thursday has had two significant provisions added since the first outline was published. The principal one is retroactive repeal of the recapture of the excess earnings clause of the Transportation Act of 1920. Justifiable in its basic principle, the clause proved to be unworkable and costly as a practical matter. A large part of the railroads' present difficulties is due to its presence on the statute books. By limiting the return upon railroad stocks even in prosperous times to 5½ per cent, this clause of the act depressed the value of those stocks and encouraged borrowing money by bond issues, and left the financial structure of nearly every railroad top-heavy with bonds—as has always been the case, unfortunately. Further, it discouraged economies in operation. If the Administration's bill does nothing other than lift this millstone from the neck of the railroads, it will help materially in laying the groundwork for a better future.

Railroad Holding Companies

PLACING railroad holding companies under the control of the Interstate Commerce Commission is the other significant proposal added to the Administration's railroad bill. At present the I.C.C. can order one railroad company to divest itself of control of another railroad when such control is found to be contrary to the public interest, but in recent years railroad operators have found that they can effectively nullify the I.C.C.'s orders by the simple expedient of transferring the stock of the controlled line to a holding company which they also control but which is beyond Commission jurisdiction because it is not a railroad company. This means of avoiding I.C.C. control should not be permitted to continue if a constitutional way of ending it can be found. The commission has long urged it upon Congress. The pending railroad bill is a logical place to lay the foundation for this needed change.

The Spoils System

ENGINEERS who have written highway history in America during the two decades that have made the United States the greatest roadbuilding nation of the world have been led rapidly to the political executioner's block since the elections of last November. Babcock, Fortney, Henderlite, Hinkle, McWhorter, Sevison, Sheets, Slack and Titus are names that are familiar to all who know the work of engineers. The roadbuilding industry can ill afford to have such names erased from the roll of workers, and the taxpaying motor-vehicle operator will miss them from the appointed guardians of the hundreds of millions that he donates out of depression income in order that he may travel cheaply and safely by highway. It is no reflection on the executives and engineers who take over the work to say that the loss of these engineers to the roadbuilding councils of their own states and the nation will not be a light one. Neither the new incumbents nor the engineering profession in general can well approve of the politics-manufactured charges of inefficiency and extravagance brought forward to justify some of the dismissals. Had the new administrations avoided making such charges, their actions would appear more decent. As it is, they stand forth starkly as exhibitions of the worst brand of partisan politics. For the engineers who have been dismissed, their separation from the service is a distinction, not a disgrace.

Better Welds

THE ADOPTION of a new welding-wire specification by the American Welding Society is the outward manifestation of a change that has been gaining momentum in the past two or three years, namely, the use of coated wire, or wire that is covered with a flux. Coated electrodes are not yet common in structural welding work in this country, but all conditions point to their increasing popularity on the ground of the greater ductility, uniformity and strength of welds and speed in welding that they offer. Structural welding cannot long do without these advantages. Foreign engineers use coated wire almost exclusively, as is illustrated in the article in this issue on welded bridges in Australia. American engineers might also give some consideration to two other Australian practices—weld deposition in multiple runs and the use of a given amount of filler metal in a specified length. Large welds must of course be built up in layers, but Australian engineers use multiple runs for all welds over ¼ in., which means all structural welds. Homogeneous welds with maximum uniformity and penetration are said to result. Depositing a 10-gage electrode in a length of 9 in. gives a weld with a strength of 3,000 lb. per linear inch, it is said. By specifying welding on such a basis, the Australian engineers say, the necessity for requiring a certain fillet cross-section is eliminated and the risk of trying to build up too quickly is avoided.

Power on the Tennessee

SEVERAL years ago Congress instructed the Corps of Engineers to prepare a comprehensive scheme for navigation, flood control and power on the Tennessee River and its tributaries. In due course of time the report was completed and printed—and filed away in the government archives to collect dust until such time as the many schemes for federal development of more power on the river should take definite form. As that time has arrived, we have in this issue reproduced from the report the plan and profile of the river and its tributaries, the preliminary drawings for the Cove Creek and Gen. Joe Wheeler dams (now proposed to be built), and some pertinent figures as to power possibilities and costs. No attempt has been made even to summarize the entire findings of the report. That would be impossible in the space available, and most of it has no bearing upon the limited program of work now proposed. One fact should be borne in mind in studying the army

engineers' report: adhering to a long-established federal policy, the engineers of the corps laid out their system of dams to provide the maximum amount of slackwater for navigation. A navigable channel is carried all the way up the Clinch River, employing 22 locks and a high barge lift in the last 100 miles. If navigation had not been considered, it is probable that a very different layout of power dams would have been made. Thus the locations shown for many of the dams on the map and profile reproduced in this issue are not definitely fixed; they represent only a preliminary outline of development to be modified to meet changing conditions as the work progresses.

Deliberative Thinking Needed

RESORT to a committee of the Construction League to draft a uniform and equitable surety bond should offer hope for fair consideration of the questions in dispute. The committee named may be depended upon to exercise restraint in its deliberations. This has seldom been the habit of the dominating voices among either contractors or surety interests; almost every past attempt to arrive at agreement ended in dissension arising from lack of judicial temper and self-control. Unfortunately the new committee starts with the handicap of having been set up by only one party to the dispute, and of lacking legal representation—this in a case ensnarled in laws and commission regulations! It will do well to call in advisers who will fill these gaps in its organization. The problem merits all the thought the committee can give, not only because it has long been a sore spot in the construction body but also because its solution reaches into the specification practices of architect and engineer and affects the interests of the public in its capacity as purchaser of construction work.

Prices and Industry Control

IN THE three years since the depression began, industrial regulation has loomed ever larger as a necessary element of economic stabilization. Last Thursday President Roosevelt placed it definitely on the recovery program when he said to the U.S. Chamber of Commerce that our orderly industrial system has degenerated into one of the highest disorder, and that the restoration of order depends on the willingness of the units of each industry to work together "to prevent overproduction, to prevent unfair wages, to eliminate improper working conditions." He went so far as to promise the government's cooperation in dealing with obstructive minorities.

This pronouncement opens a broad and promising prospect toward a new industrial era. On the following day, however, the Secretary of the Interior greatly beclouded this fair prospect. In rejecting bids for cement for Boulder Dam on the ground that the prices asked were exorbitant, he attacked the "cement trust" and stated that he would insist on lower prices even if the government had to go into the cement business. It is evident that President Roosevelt and Secretary Ickes are not speaking the same language.

At a time when the government has strenuously endeavored to bring about a rise of prices and seeks to support and maintain this rise, it is strange to find an official denouncing prices as exorbitant, especially prices that must discount the events of future months. No evidence was offered by the Secretary to show that the prices were unduly high, or that there had been any agreement or understanding about them on the part of the bidders. But quite apart from these points, it is of highest importance that the essential issue be made clear, so that the attitude of the government can be understood and followed. If we cannot go forward into recovery in harmonious understanding of industrial and governmental policies, our progress is sure to be halting.

For two years the production and sale of cement, like that of many commodities, has been conducted at heavy loss because of low demand; in addition the cement industry has been engaged in ruinous competition, each manufacturer underbidding his competitors in the effort to get an advantage by obtaining such volume of business that his own losses would be minimized and his competitors forced to ask a higher figure on their smaller output. This is an old plan of commercial strategy, often tried in times of deficient demand and as often proved a failure. It was not notably successful in the present instance of cement, for the 1932 financial reports indicate that cement manufacturers sustained very heavy losses. Presumably it was such a condition that moved the President to speak of higher commodity prices when he asked the Chamber of Commerce to act in support of his economic reconstruction program. "We have sought," he said, "to effect a broad elevation of commodity prices." Clearly, he did not argue for maintaining prices at bankruptcy levels or desire the government to buy cement below cost.

During the past year or two standard portland cement has been sold at prices below $1 per barrel at mill, and in general far below cost—cost depending on both location and volume. For the Boulder Dam project, however, a cement of unusual qualities was demanded, and the manufacturers found it necessary to carry on elaborate experiments to determine what kind of cement would provide these qualities and how it could be made. The long researches of the Riverside Cement Co. on this point, which in part were the foundation of the Boulder Dam specifications, were reported in our columns only a few months ago (*ENR*, Oct. 6 and 13, 1932, and April 6, 1933). Such cement is most costly than ordinary cement, and its cost may go up still more if the general rise of price level continues during the long period covered by the proposed contract. Mr. Treanor's statement, quoted in our news pages, gives further facts.

Beyond question, if industry is to get back to normal health it must put an end to price wars and once more operate with some regard for cost. Under established practice it should reach this point by competing as to prices and other terms, each manufacturer setting his price as low as his expectation of fair profit might warrant. Under the coming and necessary doctrine of industrial self-regulation as endorsed by the President, each industry would cooperate, under suitable supervision to the end of serving the interests of user, worker and industry alike. Under neither doctrine, however, should it be required to operate on a losing basis. Under neither doctrine should it be necessary for the government to enter into manufacture and still further expand a productive capacity already excessive.

Early approach to restored economic health involves an understanding of the conflict of attitude between the President and the Secretary of the Interior. There is no reason why the question should long be left in doubt. The deciding points as to what constitutes adequate competition, effective cooperation of industry and fair price ought to be capable of ready determination.

NEWS OF THE WEEK

Bids Rejected On Special Cement For Hoover Dam

BIDS on 400,000 barrels of low-heat cement for Hoover Dam, received by the Department of the Interior on April 24, were rejected on May 5 by Harold L. Ickes, Secretary of the Interior. At the same time the Secretary filed a complaint with the Federal Trade Commission alleging that cement manufacturers had entered into an agreement to raise prices to the government. In a public statement Mr. Ickes said that increasing the price of cement when the government is striving to alleviate unemployment by building works must have a deterrent effect on the government program and will reduce the amount of money available for wages. He said also that the government would not pay exorbitant prices for cement but would if necessary build a mill of its own.

It is understood that the complaint filed by the Secretary with the trade commission was primarily in connection with bids to the state of Illinois for state highway work, where three successive sets of bids have been rejected as excessive. However, the complaint was made public only after the submission of bids for the 400,000 barrels of cement for Hoover Dam. These latter bids were not given out officially, but it was stated at the Interior Department that when the freight rate was added most of the bids were practically identical.

Many bids submitted

It is known that nine companies submitted bids on April 24. Four of the companies, Riverside Cement Co., California Portland Cement Co., Southwestern Portland Cement Co., and Monolith Cement Co., submitted a joint bid. These same companies submitted a joint bid last September and were awarded a contract at $1.24 per barrel, while in the April 24 bidding they are understood to have bid $1.55. Secretary Ickes feels that an increase of 31c. in so short a period, when there had been no increase in wages, cannot be justified. It is true that the April 24 bid covered low-heat cement, but Reclamation Service engineers are said to believe that a 5c. spread for the additional processing is ample. It is pointed out that in the bids awarded by Pasadena for cement for Pine Canyon dam the difference between the cost of standard cement and of low-heat cement was only 9c. It also is alleged that lower bids have been made recently on standard cement to the Union Pacific R.R. and that a large shipment of cement to Panama brought only 70 cents.

The delivered cost of the cement under the April 24 bids would be $2.41 a barrel. Based on this price for all cement for the dam it is calculated that the April 24 bid price would add $1,500,000 to the total cost of the structure, as compared to the September bids.

While Secretary Ickes feels so strongly on the subject that he is prepared to order the erection of a mill near Boulder Canyon unless bids are forthcoming at something like the price level prevailing last year, his action is criticized in many quarters as hasty and ill-advised, and not based on the full facts. It is also pointed out that his action is not in accord with the President's policy to increase commodity prices to a point where they need not be sold at a loss or at a price level that would make fair wages impossible.

Attitude of producers

Representatives of the cement industry point out that intensity of competition forces uniformity of price in cement as it does in any other common standardized commodity. Prices at any one time and place, it was stated, are usually uniform for cement for the same reason that they are uniform for other standardized products in wide and common use, such as bread, milk or gasoline. No one will pay one seller more for the same article than he has to pay elsewhere, and no seller can long hold a lower price, for competitors are quickly forced to his level if they want to get business.

It was pointed out that in some parts of the country prices increased last year. For more than a year there had been a furious price war in the cement industry. Price wars do not last long. As soon as they are over, prices as a rule snap back. All published reports of cement companies show that heavy losses have been sustained in the last two years. The warfare brought prices in 1932 to the lowest level in sixteen years.

It is contended that the attitude of the Secretary differs from that of the Administration, which has been insistent that the business situation cannot be saved and wages and purchasing power increased unless there is an advance in commodity prices. It proposes to cooperate with agriculture, industry and foreign governments to increase them. Attention was called to the fact that the Department of the Interior has been active in conferring with oil men regarding the restriction of production, the obvious result of which would be an increase in price. Much has been said in Congress as to the fairness of the 1926

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Emergency Work Commission Set Up for State of New Jersey

A bill creating an emergency public works commission has been passed by both Houses of the New Jersey state legislature. The purpose of the commission is to seek out and assist in arranging for the financing of needed public works throughout the state either through an R.F.C. loan or otherwise. The commission will be similar to the one appointed some months ago by the Governor of the State of New York to coordinate the presentation of applications for R.F.C. loans. It also will be available to give whatever assistance it can to the pending federal public works program.

Columbia Basin Development Plans Take on New Form

ORGANIZATION of a Columbia basin power and irrigation district and immediate start of work without authorization of Congress has been proposed by the Columbia basin commission following a conference of representatives of the state of Washington with President Roosevelt. Heretofore it had been proposed to have this great development scheme of the Pacific Northwest taken up as a federal project for irrigation and flood control, and bills have been introduced into Congress from time to time authorizing the work but have failed of passage.

The proposal now put forward would not require additional federal legislation. A power and irrigation district could be organized under the laws of the state of Washington and the district could apply to the R.F.C. for a loan to finance the undertaking. The state would share in the cost of the work if the state bond issue of $10,000,000 for public works is held constitutional.

Under this plan the dam at the mouth of Grande Coulee would be built to a height of 145 ft. instead of 354 as originally proposed and no provision would be made for pumping water from the Columbia up into the Grande Coulee for irrigation purposes. Elwood Mead, Commissioner of Reclamation, estimates that under this plan 250,000 kw. of primary power and 250,000 kw. of secondary power could be developed. Ultimately the dam could be raised to the full height to provide additional power and water for the irrigation of from 1,000,000 to 1,200,000 acres in south-central Washington. The limited undertaking now proposed would cost $60,000,000 as compared with $160,000,000 for the larger project.

In 1931 engineers of the Bureau of Reclamation completed a report on the Columbia basin project. An abstract of it was published in *ENR* June 30, 1932, p.907.

Transmission Line Loan Made

A loan of $22,800,000 to the Department of Water and Power of the City of Los Angeles for the construction of transmission lines to Hoover Dam was made by the Reconstruction Finance Corp. on April 29. Granting of the loan was authorized on Feb. 2. The loan covers $14,000,000 for actual construction of the 271 miles of transmission lines required to connect Los Angeles and other cities of Southern California with the power plant at Hoover Dam, and $6,000,000 for expenditures to be made on the city's account by the U. S. Reclamation Bureau in the power plant at the dam. The main transmission line will be two-circuit and will operate at 275,000 volts and will have a normal capacity of 213,000 kw.

Earthquake Reconstruction Loan

The board of directors of the Reconstruction Finance Corp., on May 2, authorized the advance of $500,000 to the Unified Rehabilitation Corp. of Los Angeles, a non-profit organization, for use in reconstruction of buildings damaged by the earthquake of March 10. The R.F.C. loan is to be at 4½ per cent and will in turn be loaned by the Rehabilitation Corp. at 5 per cent. Collateral will consist of first liens on real estate.

Rapid Transit Line on Bridge at Philadelphia Approved

Plans for the construction of a high-speed transit line across the Delaware River Bridge at Philadelphia have been approved by Governor Moore of New Jersey. Unless the plan is opposed by the state of Pennsylvania and Philadelphia, the Delaware River Joint Commission will seek a loan of $9,000,000 from the Reconstruction Finance Corporation for the work.

The plan was prepared by Ralph Modjeski, engineer for the bridge commission, and calls for the laying of tracks across the bridge in the space provided for them when the bridge was built to provide a connection between the Philadelphia subway system and a rapid transit terminal in Camden, N. J.

Conditions Laid Down for Loan For Two Bridges to Grand Island

Conditions to be fulfilled by the state of New York, by the city of Niagara Falls and by Erie County before the Reconstruction Finance Corp. will make its authorized loan of $2,800,000 for the Grand Island bridges are as follows:

(1) The development through the State Council of Parks or Erie County Park Commission of the two state parks on Grand Island to such an extent as to reasonably invite the estimated traffic.

(2) The construction, by the state of New York or Erie County of the connecting road along the west shore of Grand Island between the parks and the improvement of the remainder of this circumferential road.

(3) The construction by the state of New York of a highway not less than 24-feet in width across the island, connecting the two bridges comprising the project.

(4) The construction by Erie County of a connecting road from the south bridge to Sheridan drive.

(5) The construction by the city of Niagara Falls, of an arterial highway, connecting the Niagara Falls boulevard with the approach to the north bridge.

A recommendation already has been made to the board of supervisors of Erie County to appropriate $250,000 to purchase from the state the right of way for the parkway developments on Grand Island.

The north bridge will have thirteen cantilever spans ranging from 165 ft. to 500 ft. and the south bridge will have five cantilever spans of from 280 to 600 ft. Both structures will have a 24 ft. roadway. As noted last week, the state department of public works will construct the substructure and will supervise the entire construction operations. The firms of George C. Diehl, Inc., Buffalo, and Waddell & Hardesty, New York, jointly, have been retained by the Niagara Frontier Bridge Commission to prepare the plans for the superstructure.

Silicosis Trial Against Contractor Ends in Jury Disagreement

THE FIRST TRIAL of a multitude of suits filed against the Rinehart & Dennis Co., Inc., contractors Charlottesville, Va., seeking damages for silicosis alleged to have been contracted on a tunnel job in West Virginia, ended in a jury disagreement. Failing to come to an agreement, the jury was discharged by Judge Eary of the Fayette County, W. Va., circuit court, ending a trial that lasted nearly six weeks.

The first case to be tried was filed by one Raymond Johnson, who was suing for $25,000, alleging contraction of silicosis in his 67 weeks of employment in the tunnel. To date more than 200 suits have been filed, aggregating over $4,000,000. It is reported that many of those claiming silicosis damages worked within the tunnel for very short periods.

The Rinehart & Dennis Co. is contractor on a hydro-electric project being built by the New-Kanawha Power Co. on the New River near Hawks Nest, W. Va. Included in the development is a pressure tunnel 16,250 ft. long, varying in size from 31 ft. to 46 ft. inside diameter. An adit was driven about mid-point of the tunnel, providing four working faces in all. Tunnel driving was started in 1930, both sections being holed through in August and September, 1931. Rock formation varied, with shales and sandstones predominating.

Plaintiff Johnson worked in the No. 1 or outlet portal heading, and all testimony at the trial was confined to conditions prevailing at this heading. All suits claim negligence on the part of the contractor in conduct of the work. Testimony on the part of the plaintiff, many of whose witnesses were fellow workmen who have filed similar suits, alleged that the contractor forced the drillers to drill dry, that ventilation was poor or non-existent, and that the air within the tunnel was intolerably dusty at all times. Local doctors and one medical authority from the outside testified that the plaintiff was afflicted with silicosis.

Many experts from the tunnel construction field testified for the defense, including rock drill and other equipment representatives, and powder service experts, all of whom had close contact with the work. All of the defense witnesses testified that drilling was never done dry, but that the latest type water leyners was used exclusively, always with water on the drills. Ventilation experts claimed the tunnel was unusually well ventilated, and that the contractor installed fan and ventilation duct equipment of greater capacity than was thought necessary by manufacturers of this equipment. The state mine inspector testified that the ventilation installation exceeded his recommendations. An outstanding medical authority on silicosis testified that X-rays taken of the lungs of the plaintiff revealed symtoms of pulmonary tuberculosis, but no silicotic condition.

All testimony presented by the defense tended to show that the contractor carried on the work in the best accepted manner, and was paying close attention to the health and safety of the workmen. The accident record on the job is good.

License for Big Power Project in South Carolina Extended

The Federal Power Commission, on April 22, amended the license of the Columbia Railway & Navigation Co. for its large hydro-electric power project on the Santee and Cooper rivers in South Carolina to extend the date of completion of the project to Dec. 31, 1936. The original license granted in 1923 set a completion date at the end of 1928 but because of changes required by the federal government and modifications proposed by the company which were not approved by the Federal Power Commission the company was unable to carry on the work at the rate scheduled in the original application.

The project includes the construction of a diversion dam on the Santee River and a dam in the Cooper River valley near Pinopolis to form a reservoir into which water from the Santee River would be diverted through a canal, also a power house and tailrace to discharge the water into the Cooper River. Storage of 1,254,000 acre-ft. would be provided in the reservoirs and the power plant would have a potential capacity of 200,000 hp. The cost is $34,000,000.

Railroad Legislation Expanded

The Administration's emergency railroad legislation, as presented to Congress on May 4, contains two important provisions in addition to those contained in the preliminary draft of the legislation outlined in these pages last week. The bill calls for retroactive repeal of the recapture provisions of the Transportation Act of 1920 and extends the jurisdiction of the Interstate Commerce Commission to include holding companies for railroad securities.

Registration Law in Minnesota Strengthened by Amendment

An amendment to the registration law for architects, engineers and land surveyors in the state of Minnesota, which had been before the legislature since January, was passed during the closing session and has been approved by the Governor. The registration act passed in 1921 has been largely ineffective, but has been kept alive in the hope that it would be improved and strengthened by amendment.

Under the amendment recently approved, no person can practice or offer to practice as an architect, engineer or land surveyor in Minnesota until he has submitted evidence of his qualifications to the registration board and has been registered. Heretofore registration was

not obligatory except for those desiring to use the term "registered architect" "registered professional engineer," or "registered land surveyor." The amendment also provides that only registered architects, engineers or land surveyors shall be employed in responsible charge of public works, and no plats of subdivisions may be recorded except by a registered land surveyor. The act does not apply, however, to elected officials.

The act does not apply to single-family homes or to two-family homes or to farm dwellings or buildings nor to buildings costing under $7,500. It permits contractors or builders to prepare plans but does not permit them to use the word "architect" or "engineer."

Registration fees for architects and engineers have been reduced from $25 to $10 and from $10 to $5 for land surveyors. The bill provides that the board of registration may make reasonable rules for classifying engineers in divisions according to their qualifications and practice. The renewal fees for architects and engineers are set at $3 and at $1 for land surveyors.

The amended law provides for reciprocity with registrants of other states and exempts architects, engineers and land surveyors in the employ of the federal government. Corporations or partnerships may engage in architecture, engineering or land surveying provided the person in charge is registered.

Transportation and Modernization Discussed by Chamber of Commerce

AT THE four-day meeting of the Chamber of Commerce of the United States in Washington, D. C., May 2 to 5, resolutions were passed by the full chamber advocating budgetary control of state and local expenditures, government support of railroad credit, regulation of motor buses, limitation of motor vehicle taxes to road improvement and opposing government operation of the Muscle Shoals properties. Sessions on national transportation policies and on property maintenance and improvement were directed particularly to highway officials and construction interests.

Rail and highway competition

Action was taken requesting the board of directors for a referendum on the report of its special committee on competing forms of transportation which recommends that both common and contract motor carriers be required to publish and adhere to their rates and that such rates should be reasonable and not discriminating. The same report also recommends that there should be standard uniform requirements in all states as to allowable height, width and length of single and combined units, axle and wheel loads and speeds as recommended by the Association of State Highway Officials.

Enactment by Congress of legislation requiring registration and regulation of interstate common carrier buses was advocated by resolution on the premise that the need for such action is recognized by the operators of such buses themselves. The meeting referred to the board of directors for consideration and appropriate reference topics such as the share of highway costs that should be borne by various classes of vehicles.

The round-table conference on national transportation policies resolved itself into a debate between R. V. Fletcher, general counsel of the Association of Railway Executives, and T. R. Dahl, vice president of the White Company, Cleveland, that was prefaced by the declaration of Rep. Sam Rayburn, chairman of the House Committee on Interstate Commerce that in the field of transportation, no group should have an advantage.

Mr. Dahl contended that the motor carriers now pay to the federal and state governments the same rate of taxes on their property and incomes as do the railroads. With respect to the allocation of highway costs to motor vehicles, Mr. Fletcher rejected the findings of the U. S. Bureau of Public Roads on the ground that it is "motor-minded." Mr. Dahl replied that until better facts can be adduced, those presented by the Bureau of Public Roads must be accepted.

Property maintenance

Current interest in movements for stimulating construction by extensive renovation of structures and plant was evidenced by an attendance of over a hundred at the session on property maintenance and improvement. In general federal aid for financing was the keynote of the four addresses on building renovation and the one on renovation of industrial plant. An opposing note was sounded in two or three discussions from the floor but this was drowned out by the volume of the cry for federal credit extension in sums ranging from a modest fifty million to a round billion of dollars. The ways of administering these credits were as many as the amounts desired.

Describing again the much described renovizing campaign of Philadelphia, Frank Parker, Professor of Finance, Wharton School of Finance and Commerce, University of Pennsylvania, Philadelphia, Pa., reached the inspiring conclusion that: Based on the success of renovizing campaigns in 100 communities of record, it is reasonable to assume that, without any extraordinary effort to finance impoverished property owners, the simultaneous development of renovizing campaigns in all cities of 25,000 and greater population would develop $1,000,000,000 additional business. Extended to cities of 5,000 people the business developed would be $2,000,000,000.

Speaking on the subject of rehabilitation of production equipment, H. Meese, Western Electric Co., Baltimore, Md. advocated liberalization of the R.F.C. act to allow loans for rehabilitation of factory equipment 55 per cent of which, he stated, is now ten or more years old.

In the general discussion following the formal address, E. T. Peterson, editor, *Better Homes and Gardens*, Des Moines, Iowa, described a movement being made to secure a billion dollars federal aid act to finance home rehabilitation, the loan to be repaid in installments over a term of years. Oscar Monrad, secretary, Association of Commerce, Elmira, N. Y., took the position that home and business building renovation, in small cities at least, did not need to go outside for money. He cited his own and other New York cities as successful examples. Richard B. Watrous, secretary, Chamber of Commerce, Providence, R. I., described a statewide renovizing campaign, modeled on that of Philadelphia, about to be started in Rhode Island. This would rely on self-financing by the owner. Various others described special procedures adopted to create public interest in rehabilitation.

Wholesale Turnover of Indiana State Highway Engineering Staff

As of May 1, William J. Titus, for eight years chief engineer, Indiana State Highway Commission, has been relieved of his duties by the governor. A. H. Hinkle, chief engineer of maintenance has also been dismissed and a number of other dismissals in important positions particularly in the field staff have taken place.

Fred Kellam, for eight years assistant chief engineer in charge of the bridge department has been named acting chief engineer. J. T. Hallett, assistant chief engineer in charge of road location, design and construction, retains the same title with somewhat different duties. P. D. Miesenhelder is to remain as head of the department of materials and tests. C. W. McClain, district maintenance engineer, has been promoted to be in general charge of all maintenance in place of Mr. Hinkle but reporting to Mr. Kellam.

Surety Bonds and Practices To Be Subject of a New Study

Because of the general dissatisfaction within the construction industry over present forms of surety bonds and the practices which have developed in connection with surety bonding, the Construction League of the United States, at the request of the Associated General Contractors of America, has appointed a committee to study the purposes of suretyship on construction contracts and to develop a form of surety bond to fully protect those entitled to protection as well as to protect public interest. The committee is made up of the following: William Stanley Parker, Boston, Mass., representing the American Institute of Architects, A. J. McKenzie, San Antonio, Texas, Associated General Contractors of America; H. C. Whitehurst, Washington, D. C., American Road Builders Association; Malcolm Pirnie, New York, N. Y., American Water Works Association; John P. Hogan, New York, N. Y., American Society of Civil Engineers; W. W. Horner, St. Louis, Mo., American Society of Municipal Engineers.

Bids on Low-Heat Cement for Hoover Dam Rejected

(Continued from p. 602)

level. Cement prices are far below that level, and below the level of price that existed immediately prior to the recent price war. Cement prices still are on an unprofitable basis, producer state.

John Treanor's statement

In answer to a request of *Engineering News-Record* for a statement of facts on the Boulder Dam bids, John Treanor, president of the Riverside Cement Co., Los Angeles, which company developed the low-heat cement and was one of the four joint bidders on April 24, says:

"Secretary Ickes must have been misquoted in the newspapers. The recent bids were not upon ordinary portland cement but upon special low-heat cement to be furnished under exacting specifications peculiar to this job. (See Woods-Steinour articles, ENR Oct. 6, 13, 1932 and Apr. 6, 1933.) My company has spent several years and many thousand dollars developing this special cement, and we placed complete information concerning it at the government's disposal without charge. These special low-heat specifications involve additional cost to manufacturer and considerable manufacturing and commercial risk not incurred in making ordinary commercial portland cement. The specifications also require unusual and burdensome storage conditions peculiar to this job.

"Recent prices were not, as stated in the Ickes interview, identical or uniform but covered a wide range of both factory and delivered prices. The last prior bidding was not two months ago, as stated in the interview, but on Sept. 26, 1932, and was for ordinary commercial portland cement. These bids likewise covered a wide price range. We are now furnishing somewhat similar low-heat cement to Pasadena for its Pine Canyon dam, but this cement is cheaper and easier to make than under Hoover Dam specifications. The price to Pasadena, submitted July 26, 1932, was $1.50 per barrel, at mill; the April 24 bid to the government on its more severe specification was $1.55 at the mill.

"Further, the recent call for bids for Hoover Dam cement contained no time limit for taking the cement, which might extend over many months, involving serious contingencies of increased manufacturing costs, especially in view of the government's inflation policy.

"We know from experience that the Hoover Dam engineers and the entire Reclamation Bureau desire to be fair and are always willing to take practical circumstances into consideration. A serious misunderstanding concerning the recent bids undoubtedly exists temporarily but I am convinced it will be satisfactorily cleared up. Secretary Ickes' interview is so full of factual mistakes that I am convinced the Secretary was misquoted."

The Illinois bids

Discussing the question of current and recent bid prices more broadly, including the Illinois bids involved in Secretary Ickes' complaint to the Federal Trade Commission, a leading cement authority states that these bids, made in answer to three successive calls, and followed by rejection of all bids, are lower than the prices of any year since the war except last year and 1931, when cut-throat competition between mills drove prices down to a ruinous level. In view of the imminent price inflation, says this authority, bid prices are too low rather than too high, as costs are almost certain to increase; bidders cannot increase the price to the state, but they guarantee the uniformity of bid prices.

Discussing the uniformity of bid prices on cement, this manufacturer points out that except in times of rising or falling market the price of a standardized commodity like cement is bound to stabilize at a uniform level, no producer asking more than the current market and no one being willing to risk bringing down the market by underbidding.

The cement price war resulted in grave losses to cement producers in spite of drastic reductions in salaries and wages and reduction in personnel. These losses have been estimated in excess of twenty-five million dollars (*ENR* Apr. 7, 1932, p 513).

SOCIETY CALENDAR

AMERICAN SOCIETY OF CIVIL ENGINEERS, annual convention, Chicago, June 27-30.

AMERICAN SOCIETY FOR TESTING MATERIALS, annual meeting, Chicago, June 26-30.

AMERICAN WATER WORKS ASSOCIATION, annual meeting, Chicago, June 12-16.

NATIONAL COUNCIL OF STATE BOARDS OF ENGINEERING EXAMINERS, annual convention, Chicago, June 26-28, 1933.

SOCIETY FOR THE PROMOTION OF ENGINEERING EDUCATION, annual meeting, Chicago, June 27-30.

ENGINEERS WEEK, Century of Progress Exposition, Chicago, June 25-30.

NEW ENGLAND ROAD BUILDERS ASSOCIATION, at its annual meeting held in Boston, elected Charles F. Knowlton, president, Simpson Bros. Corp., as president of the association and Louis R. Perini, vice-president. Joseph A. Tomasella and Frederick Hoitt were re-elected treasurer and secretary, respectively.

MID-SOUTH, American Society of Civil Engineers, at its annual meeting held at Vicksburg, Miss., May 5-6, elected Major T. B. Larkin, district engineer, U. S. Engineer Corps, Vicksburg, president, to succeed Roy E. Warden of Little Rock, Ark. The following vice-presidents were elected: W. D. Dickinson and W. W. Zass, representing Arkansas; Lieut. Herbert D. Vogel and H. W. Nugent, representing Mississippi; C. E. Boesch and Vernon H. Smith, representing Tennessee. C. H. Schwartz, Memphis, was elected secretary-treasurer.

Brief News

FOR SURVEYS OF THE WATER RESOURCES of Long Island, the state of New York has appropriated $2,000 to be used in connection with the surveys carried on by the United States Department of the Interior in Nassau and Suffolk Counties and the city of New York. The state is required to match federal funds for such survey work.

INCORPORATION of the Thousand Islands Bridge Co. for the construction of a highway bridge across the St. Lawrence River from Hill Island to Wellesley Island to connect with a proposed bridge from Wellesley Island to the New York State mainland is before the Canadian Parliament. Recently the New York State legislature set up the Thousand Islands Bridge Authority to construct the section of the proposed bridge on the New York State side of the international boundary.

THE FIRST TOW OF BARGES to go from New Orleans up the Mississippi and Illinois Rivers to Chicago has been ordered to leave New Orleans on May 31. It is expected to reach Chicago on June 22 and will mark the opening of the Gulf to Great Lakes Waterway.

Personal Notes

G. ARTHUR POOLE, former city engineer of Rochester and until recently city manager, will again head the city engineering department under the reorganization of the Rochester city government.

A. P. GREENSFELDER, president of the Fruin-Colnon Contracting Co. of St. Louis, was elected a director of the United States Chamber of Commerce at its annual meeting in Washington May 3.

MARK R. WOODWARD, assistant engineer of the Lehigh Portland Cement Co., has joined the cement equipment division of the Babcock & Wilcox Co. He will make his headquarters in the Chicago office of the company.

FRANK E. WEYMOUTH, general manager and chief engineer of the Metropolitan Water District of Southern California, has been compelled to give up the active direction of that work temporarily on account of a severe illness caused by overwork.

L. A. PETTUS, engineer for the Hydraulic Press Brick Co., St. Louis, formerly in the department of bridges and buildings of the city, has been appointed division engineer of the department of street paving and sewer construction.

GEORGE C. WARD, president of the Southern California Edison Co., Los Angeles, was made an honorary member of the Los Angeles Engineers Club on May 2. Mr. Ward has taken active part in much of the construction work carried on in Southern California in recent years.

A. W. GALBREATH, St. Louis, Mo., has been appointed advisory engineer for the Reconstruction Finance Corp. in the St. Louis area to succeed Baxter L. Brown, who recently resigned that position upon his appointment to president of the Board of Public Service of St. Louis.

W. E. BALDRY, for the past 16 years city engineer of Topeka, Kansas, was removed from that position on May 1 as a result of the change in political conditions which has taken place in the city. Mr. Baldry has as yet made no definite plans for establishing new connections.

WILLIAM B. STOREY, president of the Atchison, Topeka & Santa Fe Railway since 1920, has resigned from that position in order to retire from active service. He will remain on the board of directors of the railroad. Mr. Storey entered railroad service in 1876 as an axman on the Central Pacific and continued in the engineering department of that railroad, the Southern

Pacific and the Santa Fe, becoming chief engineer of the latter in 1906 and vice-president in charge of construction in 1910. From 1918 to 1920 he served as general manager of the U. S. Railroad Administration.

THOMAS F. GIBSON and ARNOLD BOWEN have organized the Gibson-Bowen Co., contractors and engineers, with offices at 1505 Race St., Philadelphia, Pa. Mr. Gibson, a graduate civil engineer, formerly practiced as a contractor and engineer under the name of Thomas F. Gibson Co. Mr. Bowen formerly was vice-president and general superintendent of Cramp & Co., general contractors.

Obituary

CHARLES S. DENMAN, general manager of the Des Moines waterworks, died at his home there on May 4, age 71 years. Mr. Denman had been connected with the Des Moines waterworks for more than 40 years and had served as its general manager since municipal operation began in 1919. For many years he had been active in the American Water Works Association.

DANIEL KERN, general contractor, Portland, Ore., died at his home there on April 27, age 75 years. Mr. Kern had been a resident of Portland since 1878 and had been actively associated with railroad construction and harbor work in the Northwest for many years. Among the notable work carried out under his direction was construction of the Columbia River jetties.

JEROME NEWMAN, 71, consulting engineer and former chief engineer of the California State Board of Harbor Commissioners, died at his San Francisco home May 1. With the harbor commission he was in charge of design and construction of piers and ferry slip projects carried out under a $9,000,000 development program in 1912. In 1915 he was appointed chief engineer for the commission but resigned a year later to enter private consulting practice. He graduated from the civil engineering department of the University of California in 1885 and later studied in Germany.

FRANK H. JOYNER, 71 years of age, former road commissioner of Los Angeles County and at one time division engineer for the Massachusetts highway commission, died at his home in San Marino, Calif., April 11. Mr. Joyner was born in North Egremont, Mass., and entered the service of the Massachusetts commission soon after completing his education. He remained with the commission until 1911 when he was secured by Los Angeles County to become chief engineer for its original good roads system. Later he became road commissioner for the county, remaining until 1919 when he retired to act as a consultant on highway work for various organizations including the California state highway commission.

Engineering Contracts and Capital

HEAVY engineering construction contracts reported for the past week fell to $8,609,000 one of the lowest weekly totals of the year, following a record high total of $54,000,000 for the previous week. There were no large contracts let during the week, though several large projects were announced as being in the planning stage. Public works awards amounted to $5,047,000, of which $1,313,000 was for federal work. New buildings were strongest of any class of public work, awards totaling $2,452,000. Highway lettings aggregated $1,381,000, slightly above the previous week's total for this class of work, but far below the usual volume of new highway work for this time of year. Private contracts amounted to $3,562,000, less than half the previous week's total. Contracts for the past four weeks have averaged $20,449,000. A year ago they averaged $26,936,000.

Bids will be received June 1 for the Rip Van Winkle bridge to be built by the New York highway department across the Hudson River near Hudson, N. Y. Plans for four large projects were announced during the week. These are: intercepting sewers, Toledo, $1,200,000; apartments, Toledo, $7,000,000; and apartments, New York, $2,800,000.

Including advances by the R.F.C., new capital issues for the week reached the highest total of the year, $26,893,000. The R.F.C. advanced $3,000,000 to the New York State Bridge Authority for the construction of the Rip Van Winkle bridge, and $22,800,000 to Los Angeles county for the construction of a power line to Hoover Dam. Both of these loans had been approved some time ago. The remainder of the week's productive capital issues were all public bond sales, no private financing of new character being reported.

Contracts and Capital

CONSTRUCTION
(Thousands of Dollars)

	Weekly May, 1932	Average Four Weeks	Week May 11 1933
Federal government	$6,530	$1,038	$1,313
State and municipal	13,160	14,652	3,734
Total public	$19,510	$15,690	$5,047
Total private	7,426	4,758	3,562
Week's total	$26,936	$20,448	$8,609
Cumulative to date:			
1932		$425,657	
1933		325,936	

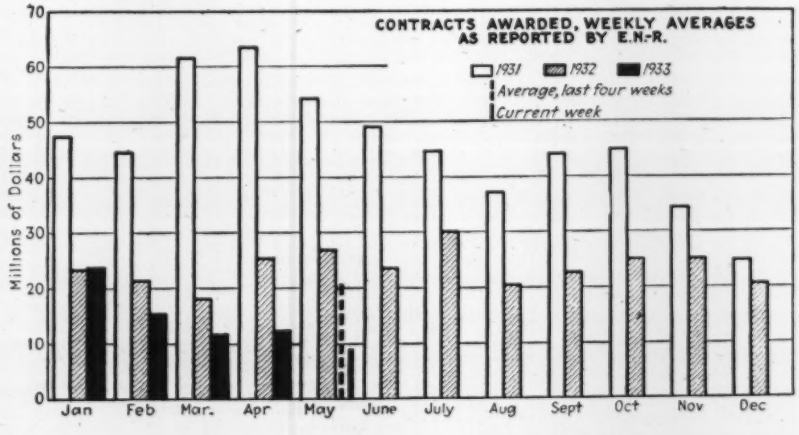
NEW PRODUCTIVE CAPITAL
(Thousands of Dollars)

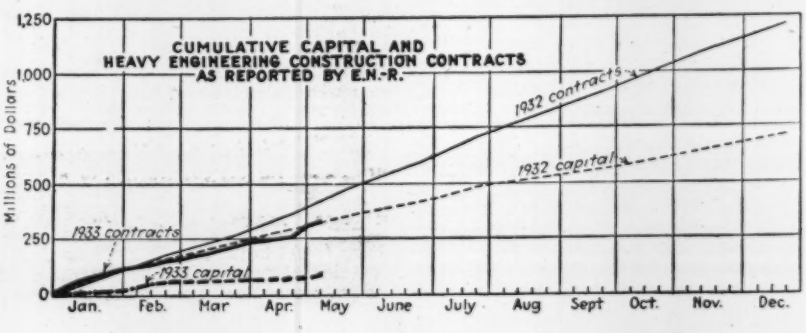
	Weekly May, 1932	Average Four Weeks	Week May 6, 1933
New capital issues	$18,000	$7,952	$26,893
Cumulative to date:			
1932		$313,000	
1933		102,674	

R.F.C. self-liquidating loans:	
Approved current week	
Approved to date (including 1932)	$204,682
Actual loans made (including 1932)	46,258

ENR Cost and Volume Index

E.N.-R. Cost		E.N.-R. Volume	
May, 1933	**164.39**	**April, 1933**	**85**
April, 1933	160.16	March, 1933	62
May, 1932	152.78	April, 1932	135
1932 (Average)	156.97	1932 (Average)	127
1931 (Average)	181.35	1931 (Average)	220
1930 (Average)	202.35	1930 (Average)	260
1913 Average	100		



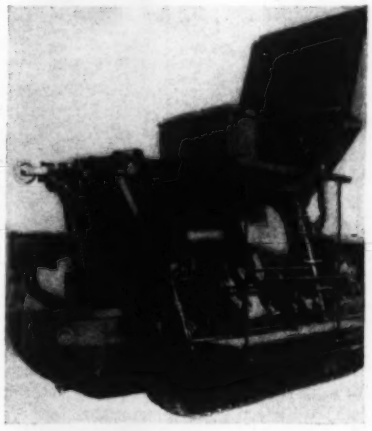


Construction Equipment and Materials

Large-Capacity Paver

A paving machine of large capacity although using batching equipment of standard size has been introduced by the Ransome Concrete Machinery Co., Dunellen, N. J. The paver has two standard size drums combined with a discharge chute between them. The materials pass from the first drum to the second. All operations are positively synchronized by automatic control.

A batch is mixed half the specified time in the first drum and the other half in the second. The design of the machine is such that the mixing action is the same when the concrete is passing from one drum to the other as when it is all in one drum.



Ransome dual-drum paver

Over-all dimensions of the machine are the same as on the Ransome standard 27-E, except as to length. Power loader, boom bucket, water tank and crawler construction are the same except that a boom having 25 ft. spreading reach is used and a 4-in. water line is provided from the measuring tank to the drum.

The manufacturer states that operating side by side on the same job with a standard machine, the dual-drum paver with the batch meter set for 1¼-minute mixing time placed 724 cu.yd. against the standard machine's 478. Three additional men were required in the mixing crew.

Business Notes

THE AMERICAN CABLE CO. has established a district office in Houston, Texas, under the management of C. A. Perryman.

THE INCINERATOR ENGINEERING CORP., 101 Park Ave., New York, N. Y., has acquired the rights of the Balmer System Incinerators. Dr. Robert Balmer will continue to serve the corporation in his former capacity. D. Colasano is president of the new corporation.

PRE COTE CORP. has moved its offices from 708 Continental Bank Building, Indianapolis, Ind., to 318 Reprblic Bank Building, Dallas, Texas. Pre Cote Laboratories, including research and technical departments, are retained in Indiana under the corporate title of Pre Cote Chemicals, Inc., and are located at 2917 Northwestern Ave., Indianapolis.

New Equipment in Brief

Electric Trucks. The Crescent Truck Co., Lebanon, Pa., announces a new electric platform truck developed for use in small plants for loads run on an average of about 2,000 lb. The platform is 58x33 in. and is at a height of 19 in. above the floor. Wheels are rubber-tired, 15 in. in diameter.

Culvert Cleaner. An adjustable culvert cleaner is announced by the Linendoll Manufacturing Co., Norwalk, Ohio. It consists of four steel radial arms hinged to a center rod at one end and connected to an adjustable cutting plate at the other. The plate is adjusted to fit the size of the pipe being cleaned.

Portable Compressor. A portable engine-driven air compressor weighing only 77 lb. complete has been announced by the Homelite Corp., 55 Riverdale Ave., Port Chester, N. Y. The engine is rated at 3 hp. and the compressor has a capacity of 13½ cu.ft. per minute, with pressures ranging up to 200 lb.

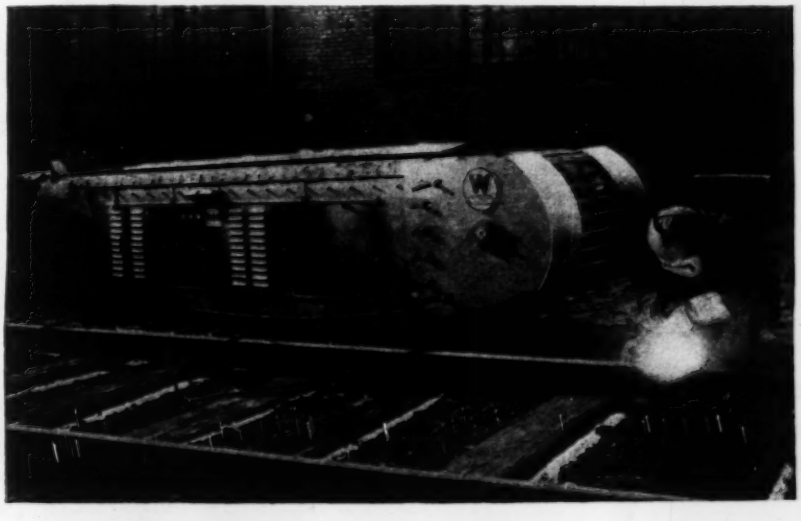
Comparator. Hellige, Inc., New York, N. Y., announces its new Aqua tester, an apparatus for facilitating the routine analysis of water, sewage and industrial wastes. Permanent color glass standards arranged in a circular frame are used with the apparatus. By revolving the disk the proper standard is brought into the field of observation.

Tractor-Type Track Welder

The Westinghouse Electric & Manufacturing Co., East Pittsburgh, Pa., recently made for the Lehigh Valley Railroad six unusual tractor welders. Developed to fill the need for a mobile power supply unit which does not require frequent transferring to and from the track for movement, this welder eliminates the use of long cable leads and does not interfere with traffic or signal operations. It is capable of operating along the shoulder of the track entirely clear of trains, can ascend slopes up to about 35 deg. and is stable in operation on side slopes of 45 deg.

The over-all width of the tractor is 30 in., allowing the unit to travel along the shoulder or between the rails. The machine measures 36 in. in height from the ground to the top of the upper tread, permitting traffic clearance with the welder on the shoulder of the track.

The tractor structure consists of a supporting steel framework, the sides of which are inclosed with heavy steel plate having suitable openings for ready access to generator controls, gasoline engine, etc. A bulkhead is built into the rear of the tractor frame on which two propulsion motors are mounted. This bulkhead, motors, worm gearing and tread sprockets are removable as one unit, thus facilitating maintenance. The control scheme permits welding and grinding operations to be carried on simultaneously while the tractor is stationary.



Westinghouse tractor welder for track work

New Publications

Torch Welding and Cutting. The Linde Air Products Co., New York, N. Y., has published an 8-page bulletin entitled "How to Figure Oxwelding and Cutting Costs." The publication shows how to set up methods of figuring costs of welding and cutting by the use of the oxy-acetylene blowpipe.

Crusher. A 10-page bulletin describing the Telsmith Gyrasphere crusher has been published by the Smith Engineering Works, Milwaukee, Wis. The crusher has a spherical steel head affixed upon an alloy steel shaft. The shaft and head are gyrated by an eccentric. The major portion of the crushing action is accomplished by the upward thrust of the head.

Welded Pipe Fittings. An 8-page booklet entitled "Designed Piping" has been published by the Taylor Forge & Pipe Works, Chicago, Ill. The booklet contains a reprint of a paper presented at the joint meeting of the New York Section, American Welding Society, and the Petroleum and Power Division, American Society of Mechanical Engineers, by F. S. G. Williams of the Taylor Forge & Pipe Works.

Road Graders. Seven bulletins featuring and describing in detail the new Austin graders have been made available for distribution by the Austin-Western Road Machinery Co., Chicago, Ill. The equipment described by the bulletins consists of the No. 12 hydraulic-control grader, No. 12 hand-control grader, No. 10 hydraulic-control grader, No. 10 hand-control grader, No. 8 grader, model 20 single-drive motor grader, and model 20 full dual-drive motor grader.

Current Construction Unit Prices

Timber Wharf, Albany Port District

THE ALBANY PORT DISTRICT COMMISSION, Albany, N .Y., is building a timber wharf on the Rensselaer side of the Hudson River as part of the port development. Contract for the wharf was let in February. Twenty bids were submitted, low bid totaling $87,623, high bid amounting to $147,575. Following are complete unit prices of the three low bids. *A*—Dennis B. Riggs, Troy, N. Y. (contract); *B*—Arthur McMullen Co., New York; *C*—Daniel Haberman, New York. Arnold G. Chapman is chief engineer of the Port District.

Item	Quan.	Unit Prices A	B	C
Excavation Cl. A	12,000 cu. yd.	$0.20	$0.25	$0.25
Excavation Cl. B	47,000 cu. yd.	.11	.19	.16
Riprap	4,200 cu. yd.	1.95	2.30	2.00
Treated piling	80,000 lin. ft.	.39	.44	.47
Untreated piling	6,100 lin. ft.	.25	.33	.43
Untreated oak piling	5,100 lin. ft.	.53	.36	.55
Treated sheet piling 6x10 in.	29,000 lin. ft.	.31	.28	.36
Treated timber	300 Mfbm.	57.50	65.00	64.00
Untreated pine decking	100 Mfbm.	35.00	40.00	57.00
Untreated pine basking log	31 Mfbm.	45.00	58.50	61.00
Iron	83,000 lb.	.04	.035	.05
Bollards	5 ea.	25.00	56.00	60.00
Cleats	10 ea.	25.00	12.00	15.00
Remove old piles	lump	100.00	320.00	150.00
Sewer line, 18 in.	lump	725.00	750.00	950.00
Water line, 2 in.	lump	180.00	260.00	120.00
Railroad track	1,200 lin. ft.	.50	.15	.65
Total		$87,623	$98,888	$105,779

Dredging, Niagara River East Channel

THREE BIDS were received by the U. S. Engineer Office, Buffalo, for the dredging of 200,000 cu. yd. of material other than ledge rock and the removal of 107,400 cu. yd. of ledge rock from the East Channel of the Niagara River between North Tonawanda and Niagara Falls, N. Y. Work is under the direction of Lt.-Col. M. C. Tyler. Following are complete unit prices. Contract was awarded in March to low bidder. *A*—Great Lakes Dredge & Dock Co., New York, (contract); *B*—Bay State Dredging & Contracting Co., Boston; C—Dunbar & Sullivan Dredging Co., Detroit; *D*—Engineers estimate.

Item	Quan	Unit Prices A	B	C	D
Excavation, material other than ledge rock above 13 ft. below m.l.w., cu.y.	196,720	$0.33	$0.40	$0.63	$0.70
Excavation, material other than ledge rock between 13 and 15 ft. below m.l.w., cu.yd.	4,320	.165	.20	.315	.35
Excavation, ledge rock above 13 ft. below m. l.w., cu.yd.	54,400	2.20	6.60	2.176	3.24
Excavation, ledge rock between 13 and 15 ft. below m.l.w., cu.yd.	53,010	1.10	3.30	1.088	1.62
Derrick hire, hours	700	33.00	7.50	17.50	27.50
Diver hire, days	17	35.00	30.00	26.00	54.27
Total		$267,316	$619,285	$314,036	$421,213

Concrete Bridge and Earth Dam, Michigan

CONTRACT was awarded late in January by the Wayne County, Michigan, Road Commission for the construction of a reinforced concrete bridge, concrete spillway and earth dam on Newburgh Road over the Rouge River. Fifteen bids were submitted, ranging from $37,334 to $90,557, though fourteen of the bids were under $64,000. Contract was awarded to low bidder. The project raises the level of the old road by building of an earth fill dam, with the new bridge spanning a concrete spillway. Approaches to the bridge, over the new fill, are paved. Following are unit prices of the three low bidders. *A*—J. A. Mercier, Detroit (contract); *B*—Martin & Krausmann, Detroit; *C*—Blanck & Gargaro, Detroit.

Item	Quan.	Unit Prices A	B	C
Remove existing bridge	lump	$85.00	$300.00	$77.00
Channel change exc., cu. yd	9,400	.18	.14	.12
Stone backfill over drains, cu. yd	1,600	.90	.75	.50
Earth fill, dam, cu. yd	39,000	.27	.30	.25
Reinf. conc. culvert pipe, 24 in., lin. ft	336	2.50	3.00	3.05
Reinf. conc. culvert pipe, 12 in., lin. ft	240	.90	1.50	1.65
2 catchbasins and one manhole	lump	125.00	175.00	120.00
Vit. tile drains 6 in., lin. ft	700	.27	.35	.28
Concrete, cu. yd	2,000	5.60	7.35	10.50
Sluice gate 4x4 ft	lump	475.00	675.00	440.00
Pipe handrail	lump	125.00	270.00	100.00
Reinf. steel, lb	155,000	.03	.028	.03
Steel sheet piles, 7 ft., lin. ft	142	4.80	7.25	4.00
Steel sheet piles, 23 ft., lin. ft	373	12.75	19.00	14.00
Steel sheet piles, 28 ft., lin. ft	20	16.50	21.00	15.50
Total		$37,334	$44,826	$45,782

Lock No. 1, Barren River, Kentucky

THE U. S. ENGINEER OFFICE, Louisville, awarded contract in February for the construction of Lock at Lock and Dam No. 1, on the Barren River, 15 miles below Bowling Green, Ky. Following are complete unit prices of the three lowest bidders. Thirteen bids were received, ranging from $553,485 ($553,885 on an alternate) to $975,240 (same for alternate). Contract was awarded to low bidder. *A*—G. L. Tarlton, St. Louis (contract); *B*—Lowensohn Const. Co., Cleveland; *C*—J. L. Simmons Co., Inc., Bloomington, Ill.

Item	Quan.	Unit Prices A	B	C
1. Cofferdam, lin. ft	1,156	$84.00	$57.00	$75.00
2. Excav., common, cu. yd	264,500	.20	.35	.40
3. Excav., old masonry, cu. yd	25	4.00	6.00	10.00
4. Fill, cu. yd	54,900	.05	.20	.20
5. Timber piling, lin. ft	113,300	.25	.20	.25
6. Steel sheet piling, sq.ft	48,000	.95	.80	.80
7. Timber, common, Mfbm	48.3	50.00	52.50	36.00
8. Timber, oak, Mfbm	6.4	100.00	80.00	125.00
9. Stone protection, cu. yd	2,050	1.50	2.60	1.15
10. Concrete, cu.yd	40,500	6.22	6.30	6.00
11. Structural steel, lb	487,800	.043	.045	.041
12. Reinf. steel, lb	146,600	.02	.025	.022
13. Nickel steel, lb	1,800	.30	.23	.21
14. Forgings, lb	18,300	.10	.07	.12
15. Steel castings, class A, lb	150,500	.054	.06	.05
16. Steel castings, class B, lb	85,300	.08	.093	.059
17. Iron castings, lb	28,400	.08	.067	.043
18. Bolts, lb	9,500	.06	.055	.09
19. Bronze, lb	1,830	.45	.45	.43
20. Babbitt metal, lb	4,320	.10	.15	.16
21. 2-in. steel pipe, lin. ft	83	.50	2.00	.60
22. 4-in. steel pipe, lin. ft	823	1.50	1.50	1.10
23. 1-in. heavy pipe, lin. ft	8	.50	.40	.80
24. 1½-in. heavy pipe, lin. ft	516	.50	1.60	1.05
25. 2-in. heavy pipe, lin. ft	8	1.00	16.00	1.25
26. 3-in. heavy pipe, lin. ft	170	1.50	5.00	1.90
27. 4-in. heavy pipe, lin. ft	810	2.50	2.00	2.50
28. 1½-in. w.i. pipe, lin. ft	482	1.00	.65	1.30
29. 2-in. w.i. pipe, lin. ft	400	1.20	.80	1.55
30. 3-in. w.i. pipe, lin. ft	474	2.50	1.60	2.15
31. 4-in. w.i. pipe, lin. ft	304	3.50	2.50	2.75
32. 2-in. fibre conduit, lin. ft	3,430	.14	.25	.40
33. 1½-in. metal conduit, lin. ft	29	.30	.60	.40
34. 2-in. metal conduit, lin. ft	83	.40	.80	.45
35. Electrical cable, lin. ft	155	.40	.50	.40
36. Handrailing, lin. ft	126	1.60	2.00	1.75
37. Tile gages, lin. ft	53	3.00	4.00	2.00
38. 8-in. pipe drain, lin. ft	300	.30	.75	.50
39. Operating machines, mitre gates, each	4	2,150	2,000	2,000
40. Operating machines, butterfly valves, ea.	4	800	900	750
41A. Butterfly valves, riveted type	4	510	1,000	675
41B. (Alternate) same, welded type	4	410	1,000	700
42. Move 2 buildings	lump	1,500	2,000	....
43. Remove pile fender	lump	300	250	....
Total, with 41A		$553,885	$568,915	$586,410
Total, with 41B		553,485	568,915	586,510